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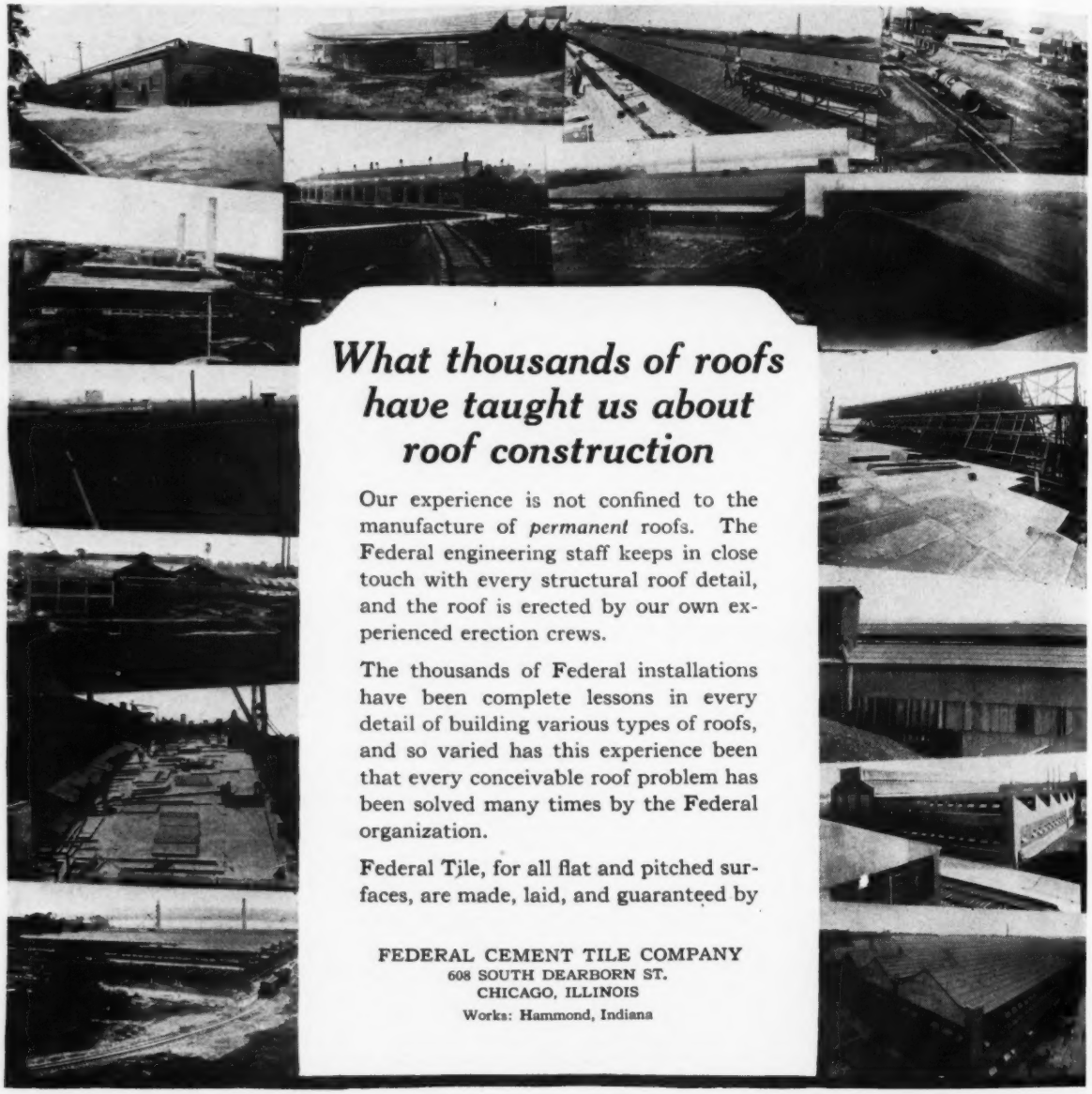

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DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

FRANK C. WIGHT
Editor

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Gone and Forgotten

ACCORDING to its lights the recent session of Congress did as well as it could. It convened in a presidential year, with a bare majority in the hands of one party and with a floating organized third party ready to throw its weight to whichever side seemed most promising. No wonder that politics and not policies prevailed. Necessary legislation, being controversial, went by the board and such laws as were able to emerge suffer most of the evils of compromise with few of its virtues. Hard as it sometimes is to realize, Congress is the voice of the people. The trouble this year is that the people are not coherent in their thinking—and the scattered and disorderly thoughts that reach the individual representatives in Washington can result only in such disorderly action as the recent Congress took. After November there may be a return to more definite mental processes.

Getting to Know One Another

INTERNATIONAL engineering congresses do not as a rule add very much to the sum total of human knowledge, but they do materially increase reciprocal understanding. Their value lies far more in the better appreciation of each other's virtues and abilities than in the actual new things in engineering that any of the delegates learn. This applies, too, to such a trip as the Pan-American Highway Commission is now making in this country, as noted this week in our news pages. These South and Central American engineers are among the most distinguished in their respective countries and, because of their standing and ability, will doubtless carry back to be put into execution, numerous excellent ideas in road construction, for they will be the first to admit that we have gone further in that work than have they. But beside that they will carry back the recollection of a brotherly welcome from American engineers and the assurance that these engineers recognize a professional brotherhood not crossed by national lines. In their turn they will, without intent, teach those Americans they meet that our Southern neighbors have engineers of training and experience quite comparable to ours who are meeting and solving problems equally important as ours. This mutual appreciation has a value in international comity far beyond the mere improvement in professional relations on practice. It is one of the primary ways toward world peace.

Voices in the Wilderness

FOR a half century engineers have advised Philadelphia to install water meters without delay. Their voices have proved to be like those crying in the wilderness. One engineer-prophet, a chief of the bureau of water, was not stoned to death but he was summarily ordered by his immediate superior to keep mouth, pen, and pencil quiet, the pencil having been used to prevent graphically the enormous water waste of Philadelphia. Twenty-five years ago engineers urged metering to bring water waste within bounds before or coincident with building filters. The advice went unheeded, except for current official or semi-official talk to the effect that the filters would be built to meet the then uncontrolled consumption and waste and would be followed by metering that would curtail waste at a rate equal to the legitimate increase in consumption through population and industrial growth, so the filter plants would suffice for many decades to come. The filters were built accordingly, at a capital outlay made huge both by the high water consumption and waste and by other considerations that have gone down into history. Metering went by the board. The filters put in full operation only some two decades ago soon became overworked. A feature of the report on meeting the water needs of Philadelphia published in our issue of June 5, p. 968, was a recommendation for metering—primarily to reduce the burden on existing filters, and thus ensure safer water, pending the introduction of a new supply. Will Philadelphia heed this renewed call to combat water waste?

Flexible Brick Road Surfaces

HARD-SURFACE roads have been so much a subject of study to improve beam strength that there is an element of strangeness in contemplating hard surfacing in which beam strength has no consideration. The brick roads of Florida generally are laid with a sand filler on a sand subgrade confined by curbs. Here is a pavement construction without beam strength. The subgrade is incompressible as long as it is confined, but the moment it shifts however slightly the brick carpet changes position. There is no power in the surfacing to bridge over a subgrade hollow. Recently C. A. Hogenthaler, an engineer of the Bureau of Public Roads, has been surveying these Florida brick-on-sand thoroughfares. Surprisingly enough, to the rigid slab pavement advocate, he finds that they have decided economic advantages. He says: "Because of the high salvage value of the brick and the small cost of relaying, the sand-filled pavements were economical and especially well adapted to a state in which uncertain development precluded a forecast of future traffic conditions. Where pavements which have been in service from 8 to 14 years, and which in that time in many cases have received little or no intelligent maintenance, have 70 per cent functioning as fair and good roads, and the remaining 30 per cent, though in need of construction, has a relaying value equivalent to 80 per cent of the new construction cost, such pavements cannot be considered as other than satisfactory." With a more rigid curb construction and a filler more stable than sand the conclusion is that flexible brick tops are a truly good road type for moderate traffic.

Grade Crossings Old and New

WHILE the old problem of highway-railroad grade crossings continues unsolved and a matter for such controversy as appeared in our correspondence columns some weeks ago, a new grade crossing problem is developing which may ultimately equal or exceed the other in importance. This is the problem of busy road intersections, where it is found that the two streams of traffic which cross each other interfere so seriously during peak hours as to cut down the efficiency of the road system. A number of such intersections have recently been studied with a view to providing grade separation as a remedy. Traffic capacity rather than safety is the objective, it will be noted. What expenditure is justifiable to secure increased capacity remains to be determined; after this the allocation of costs should not prove difficult, since even now the major cost of road improvement is borne directly by the road user—the motor vehicle operator. It is significant that this new grade crossing problem should present itself while the two existing problems of related kind, those of city street intersections and those of highway-railroad crossings, show little or no progress. In all three cases the difficulty of high cost is basic. But in the case of street as well as road intersections the solution is easy whenever the particular case becomes urgent enough to demand action; cost distribution is not a critical factor of the problem. In the matter of highway-railroad grade crossings, on the other hand, lack of progress is to be attributed mainly to the difficulty of cost distribution. The serious side of the matter is that the natural difficulty of the problem is aggravated by the traditional and still surviving antagonism of highway and railroad interests. While this condition prevails, progress is necessarily slow. In the meantime the claims of highway traffic grow steadily more insistent, and the time is rapidly approaching when it will enforce these claims whether railroad men remain hostile or not. The development of the highway intersection problem is new evidence of how the pressure of highway demand is becoming stronger.

Better Materials of Construction

MOST fields of production to satisfy the world's wants have registered a remarkable advance during the last twenty-five years. Methods and materials have been developed to a higher level, and costs have been reduced. An outstanding example is seen in automobile construction, where despite the higher cost of purchases and labor the manufacturer turns out an incomparably better product than ten years ago, at a lower price. Such advances are one measure of the world's progress.

Little or no advance of this kind is perceptible in the field of materials of construction. The quarter century has brought no change in efficiency of the major materials used by the civil engineer. Wood, steel, cement and the like are substantially the same in kind and quality as they were at the beginning of the century. Their production and distribution has come under more systematized control, with perhaps some minor influence on cost, but the materials themselves show no advance. The condition is curiously out of harmony with the continued forward movement in other lines of materials.

It may be that this absence of progress indicates that the civil engineers's materials have reached their highest point of perfection, since obviously there can be no better than the best. But this is quite improbable, and few users of any one of these materials would be ready to admit that the acme of quality and variety has been reached. It is much more likely that the arts which produced the materials of construction have been marking time in technical respects, being so much preoccupied by the routine of production and marketing that no tendencies to develop advances could make headway.

If this is the case, the production of engineering construction is distinctly out of step with other technical activities. Lack of advance in materials is bound to mean lower efficiency and higher cost of the final output as compared with other fields of production. Is it possible that this condition is related to the fact that building costs more than anything else today—that it is at a level of 1½ to 2½ times the pre-war cost, while automobiles are twice as good and cost a third less than before the war?

Improvement of quality is usually the result of two factors, competitive pressure of business and the natural seeking of men of brains to do their work constantly better. The industries which produce steels, cements, and other materials of construction were formerly well provided with men of originality, men who by instinct were always seeking for progress, and doubtless they still have men of this type. The more urgent demand from other fields, in which active progress is going on may have drawn away the larger part of this class. Yet it would seem that those who were left to the materials industries would have brought about advances if unrestrained. Evidently the pressure of manufacturing problems in a steadily growing industry was such that their influence was subordinated and their proposals suppressed.

Some recent movements suggest that the stationary period may be drawing to a close, however. The development of improved cements, the evolution of a demand for higher grades of structural steel, active study of lumber grading to give better quality and more complete utilization of the forest, are instances of a general mustering of forces for a new advance. So also is the entry of the American Society for Testing Materials into the field of research in materials a sign of a new day—a change from the twenty-year period during which that organization occupied itself exclusively with taking stock of practice in materials and defining that practice by specifications.

Both the static condition of materials production during the past quarter century and the reawakening of the spirit of improvement are attributable to some degree to the engineer, who makes use of the materials. Whenever the user needs an improved material and insistently demands it, the producing industry is very likely to supply it sooner or later; while the absence of a demand from the user will tend to encourage lack of progress by the producer. Had there been sufficiently urgent demand from the field of engineering, therefore, it is likely that advance in the quality of materials would have continued, as a response to this demand. The engineer has been largely occupied with his own problems, and remained content to accept the current qualities of materials as wholly adequate. Correspondingly, the awakening demands from the circle of users for better materials of construction has been

a direct influence in bringing about the new movement.

If the development of construction materials is about to be resumed, therefore, the civil engineer as user of these materials has a part to play in maintaining and guiding this advance. His direct participation in the work of the advance is important. By giving expression to his demands for better quality, and taking advantage of each step of advance as soon as the producer makes it available to him, the engineer will assure the certainty of progress.

Government Engineering Reform Delayed

AMONG the impedimenta that Congress left behind when it adjourned to attend the nominating conventions was the joint committee report on the reorganization of the governmental departments. The report is accompanied by a bill which, in the normal course, will be brought up at next winter's session. One more step has thus been taken in the slowly traveled road toward the more efficient administration of the departments at Washington, but the goal is yet a long way off.

Engineers can rightfully take credit for the revival of interest in this most necessary reform. Obvious as has been the necessity for a more rational coordination of departmental activities, and frequent as have been the abortive moves toward that end, it was not until the organized engineers some four years ago initiated the campaign for a department of public works that any directed effort was made to get Congress to consider the subject. That effort has been followed up to good effect in that a serious study of the needs of reorganization has been made, with fair prospect of some effective change, but unfortunately for the engineers who started the movement their original purpose has been subordinated to the broader problem. The report just rendered and the bill which accompanies it satisfy but a few of the requirements of reform of the engineering activities of the federal government.

The ideal toward which engineers look is one department of the government which will control all public work. That department would have at its head, if not as a cabinet officer at least as an executive reporting solely to a cabinet officer, an engineer of training and experience whose appointment and tenure would be guarded against political control. The engineering personnel would likewise be selected and promoted for ability, be given adequate remuneration and guaranteed tenure and retirement allowance. The organization of the department itself, that is the adjustment of functions and the allocation of personnel, need not be made a matter of law but can be left to internal direction, for it would not be more difficult than similar problems in large private business. Such a department would measurably reduce the difficulties of government engineering, work economies and raise the shattered morale of government engineers.

The present proposed scheme is far from this ideal. It approximates it only in the consolidation of a few of the major engineering activities under one head. There is no provision whatever for the more necessary assurance of a well paid, permanent and organized corps of civilian engineers, and without that no department of public works can succeed.

The joint committee report shows, too, that the Corps of Engineers of the Army has been successful so far in its fight to retain control of the river and harbor work, and to remain a separate and essentially aristocratic group outside the general body of government engineers. This, we believe, is unfortunate both from the point of view of general engineering efficiency and of the army but it is the result of two things, the strongly entrenched position of the Corps in official Washington and the lack of sympathy of the civilian engineer adherents of a public works plan with the purposes and problems of the Corps of Engineers. On the one side is the army firm in its slogan "what we have we hold"; on the other the civilians concerned only with taking away the army engineers' work and substituting therefor only the vaguest promises of engineering service. There is here an unnecessarily bitter opposition that must be compromised.

This journal has stated many times before, and it repeats now, that it believes the close corporation control of the river and harbor work of the country by the uniformed members of the Engineer Department is neither advantageous to the government, which does not get the full service that it would by a more democratic organization, nor to the army, which thereby is deprived of the broader engineering experience modern war demands. At the same time it recognizes the possibility of war, with its sequents, the necessity for an army, and an engineer corps which has been trained more, possibly, in the responsibilities of leadership and direction than in the details of engineering practice.

All plans suggested by civilian engineers have emphasized the defects of the present system of river and harbor control but have neglected the just arguments of the army in demanding some measure of responsible control. The army's fight has been made partly to retain its old authority, which it naturally clings to, and partly to resist what it considers will be complete subordination to an antagonistic civilian control. And it has managed so far to impress Congress and its committees with the justice of its fight.

The fight is by no means ended with this report of the joint committee. Both army engineers and civilians should realize that. But there is room for compromise between the two. The veiled and spoken accusations which have demeaned both sides should stop and some move be made toward a better understanding between the civilian and military branches of the profession. So long as there is effort made to abolish the Corps of Engineers as an engineering entity and disperse it among a civilian group or relegate it to purely military duties, the Corps will be able to put up a successful resistance. So long as the Corps maintains exclusive control of river and harbor work and permanently subordinates the civilian assistants, so long as its officers continue to have only this one kind of engineering experience, there will be continued effort to bring about a change. Now is the time when engineers should seek a compromise, for a new administration, whether it be that of Mr. Coolidge or of some other, comes in next March, and the time to get such reforms effected is in the first year of an administration.

The joint committee's bill, therefore, promises something better than we have had before but it is by no means satisfactory. Those who believe that engineering service is important to the welfare of the country and wish to improve that welfare by improving that service must continue to work for a better organization in the departments at Washington.

Penstock Manifolds Designed to Simplify Stresses

Two Entirely Different Types Used—Large Sections and High Heads Introduce Unusual Stresses —Deformation and Indeterminate Stresses Largely Eliminated

By H. L. Doolittle and Chester W. Larner

Unusual penstock manifolds of radically different types have been installed recently at two of the new power plants of the country, one at Big Creek, No. 3 plant of the Southern California Edison Co., and the other in Vermont at the Davis Bridge plant of the New England Power Co. Both are designed to eliminate as many indeterminate stresses as possible. The different methods of attack are interesting.

Big Creek No. 3 Penstock Manifold

By H. L. Doolittle
Assistant Construction Engineer, Southern California Edison Co.

Among the novel features of the Big Creek No. 3 hydro-electric plant of the Southern California Edison Co. is the steel manifold at the upper end of the penstock. Before describing the details of the manifold itself, its situation in relation to the rest of the plant and the conditions which control its design should be outlined.

From the diversion dam in the San Joaquin River a 21-ft. diameter tunnel, about 5.5 miles long, runs to a point directly above the power house. The lower end of this tunnel will ultimately connect with six 7 ft. 6 in. diameter penstocks. The maximum head at this point under conditions of maximum surge and high water in the river is approximately 200 ft. so it will be appreciated that the design of a manifold to connect the tunnel with the six penstocks offered many difficulties. It was

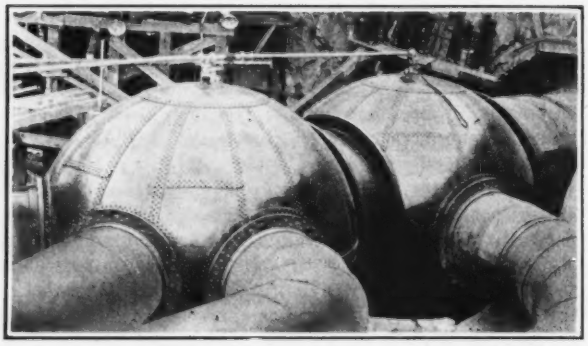


FIG. 1—EXTERIOR VIEW, BIG CREEK NO. 3 MANIFOLD

decided to install an 18-ft. diameter steel pipe in the lower 300 ft. of the tunnel in order to insure a watertight connection so that the problem to be solved was to connect the 18-ft. pipe with the six 7 ft. 6 in. diameter penstocks.

A great deal of study was put on a design similar to an ordinary branch fitting but on account of the large diameters and comparatively high pressure it did not seem practicable to reinforce the structure to take care of the very great stresses involved. It was therefore apparent that a radical departure had to be made from the common type of design used for smaller fittings of this sort.

The design finally adopted consists of two 24-ft. sheet metal spheres connected by a short length of 15-ft. pipe. The 18-ft. pipe from the tunnel and two 7 ft. 6 in. penstocks connect to one sphere and four 7 ft. 6 in. penstocks connect to the other. At the present time

FIG. 2—INTERIOR VIEW, BIG CREEK NO. 3 MANIFOLD

only three of the penstocks have been installed. This type of design has many advantages over any others that have been suggested as all stresses are determinate and easily calculated and the shop work is not difficult. A sphere under internal pressure is subjected to simple hoop tension in all its parts so that the reinforcing of an opening in the sphere is a simple matter compared to providing for the complicated stresses that exist around an opening in the side of a cylinder. A further advantage of using a sphere is that the thickness of metal required for a sphere is only half that of a cylinder or pipe of equal diameter.

The reinforcing around the openings to the 18-ft., 15-ft. and 7 ft. 6 in. pipes is quite novel. Diagram *a* in Fig. 3 shows a section through one of these openings with the stresses involved. Stress *A* is due to the hydrostatic pressure on the cross-section of the connecting pipe. The stress *B* is the hoop tension in the sphere and *R* is the resultant of *A* and *B*. It can be easily

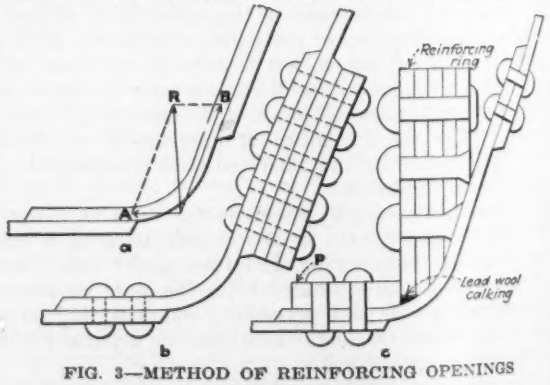


FIG. 3—METHOD OF REINFORCING OPENINGS IN MANIFOLD

(a) Stresses at opening in manifold; (b) usual method of reinforcing an opening; (c) method of reinforcing adopted.

proved that R is always at right angles to A no matter what the relation between the diameter of sphere and pipe may be.

Diagram b in Fig. 3 shows the most obvious method of reinforcing the opening in the sphere. It is seen that this has disadvantages on account of the difficulty of making the reinforcing rings of spherical shape and also making the joint watertight. The method finally adopted was to place the reinforcing ring on the *outside* of the flange directly in the line of the resultant pressure, in this position the ring is subjected to simple hoop tension. These reinforcing rings were made of several plates riveted together, the larger ones being assembled in the field. The inside of the ring is bored to provide a good bearing on the flange. In order to facilitate erection the inner edge of the ring was relieved as shown and the space between the ring and the flange calked with lead wool. The reinforcing ring was held in place with the keeper plate P.

FIG. 4—REINFORCING RING ON OUTSIDE OF MANIFOLD
Keeper ring bolted in place.

Fig. 4 shows the details of the reinforcing rings, and Figs. 1 and 2 are inside and outside views of the manifold. The manifold was made in the shops of the Chicago Bridge and Iron Works and erected under their supervision. Owing to the excellence of the shop work the erection was performed with a minimum of difficulty.

A patent on the design of this manifold has been applied for.

The Davis Bridge Penstock Manifold

BY CHESTER W. LARNER
President, Larner Engineering Co., Philadelphia, Pa.

THE three-branch penstock manifold of the Davis Bridge hydro-electric plant of the New England Power Co., although conforming in many respects to the usual type of penstock manifold, has some new features which are of interest. It not only preserves smooth contours without sudden changes of velocity, but eliminates all bending moments in the main members of the structure. It is devoid of tendency to deform when subjected to internal pressure and, consequently, it is unnecessary to bury it in reinforced concrete, as has been the practice with the usual type of construction heretofore employed for such purposes.

The inlet is 14 ft. in diameter, connected to a 14-ft. conduit, and there are three outlets, each 8 ft. in diameter connected to 8-ft. penstocks. The static pressure is 90 lb. per sq.in. Fig. 6 is a view of the manifold assembled before shipment at the plant of the Merchants Shipbuilding Corp., Chester, Pa., where it was fabricated.

The essential features of the design are shown by Figs. 5 and 7. The 14-ft. circular inlet is divided by two vertical chords, into three segments of equal areas. These three segments form the beginning of three passages of equal areas, separated only by imaginary chords and leading to the three 8-ft. diameter outlets.

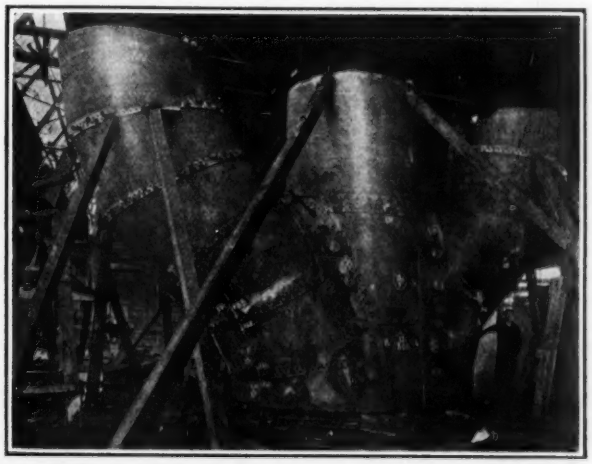


FIG. 6—SHOP ASSEMBLY OF DAVIS BRIDGE MANIFOLD

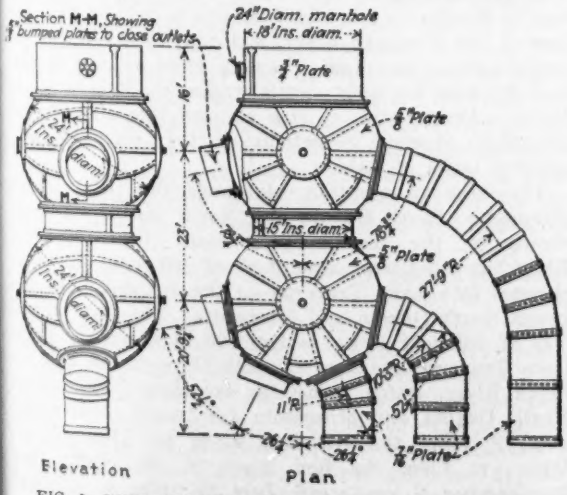


FIG. 5—DETAILS OF BIG CREEK NO. 3 MANIFOLD

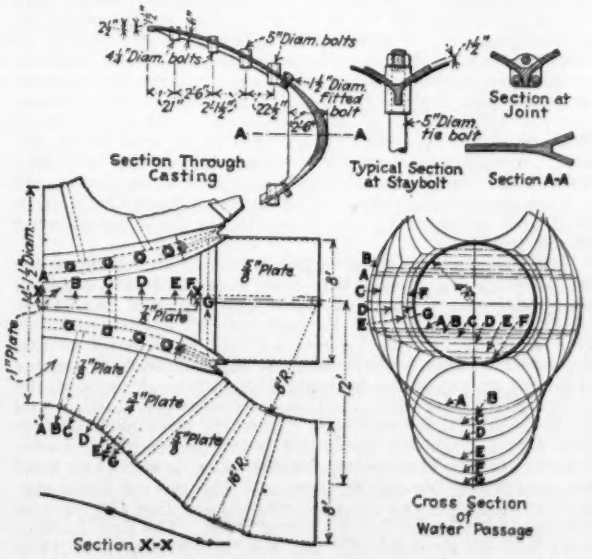


FIG. 7—DETAILS OF DAVIS BRIDGE MANIFOLD

The shapes of the three passages gradually merge from the segmental form at the inlet to the circular form at the outlet. At intervals along the dividing chords are located stay bolts, which take care of all of the forces developed in the structure, which are not taken up in the walls. The forces in the complete structure, including the staybolts, are in perfect equilibrium and, consequently, there is no deformation under pressure.

The cross-sectional form of the pipe walls, starting with a segment of a circle, and remaining segmental with a decreasing radius until they finally end with three tangent circles, is the vital feature of the design. As long as the walls are circles or segments of circles, there will be no tendency to change shape, when subjected to internal pressure, and no free forces will be developed outside of the walls except the vertical forces acting along the common chords, which are taken care of by the staybolts.

The Davis Bridge manifold employs cast steel flanges at the crotches between the branches. The staybolts pass through bosses cast on the flanges, and the plates which form the walls of the branches are riveted to the flanges.

The principles of design involved are not confined to the particular type of manifold illustrated. They are equally applicable to manifolds having outlets of unequal area, and unsymmetrical arrangements such as a series of side outlets on a pipe. The fundamental principles are of almost universal application in the construction of pipe branches, and represent an important advance in the design of such structures.

Electrification of the Virginian Ry.

THE big project of electrifying 134 miles of line on the Virginian Ry., between Roanoke and Mullens, Va., in order to secure more economical handling of the increasing coal traffic, is reviewed as follows in the railway company's annual report for 1923. This project is under contract to the Westinghouse Electric & Manufacturing Co.

The electrification includes an 11,000-volt trolley line on steel structures over 134 miles of main line track and a total trackage of 231 miles, including double track, yard tracks and sidetracks; a 50,000-kw. steam generating plant at Narrows, Va.; an 88,000-volt transmission line carried on steel towers, largely across country on independent right-of-way; seven transformer stations to step-down the current to 11,000 volts for the trolley wire, and twelve electric locomotives of three units each, together with repair shops and inspection facilities. Each locomotive will weigh 625 tons. Two locomotives (one acting as pusher) will take a 6,000-ton coal train up Clark's Gap Hill at 14 m.p.h., and one locomotive will take a 9,000-ton train from Clark's Gap yard to Roanoke at 14 and 28 m.p.h., the lower speed being on the grades.

The power plant, transformer stations, transmission line and distribution system will have a capacity for handling an annual coal tonnage of 12,500,000 tons, and the twelve electric locomotives will be sufficient to handle 8,000,000 tons annually. Other locomotives will be ordered as this tonnage increases. The scheme of electrification is capable of indefinite expansion by installing additional units at the power plant, providing additional transformer stations and using additional locomotives. Contracts have been authorized also for stacks, structural steel, power plant, transformer station equipment, transmission towers and steel for supporting the trolley line. At the end of 1923, surveys were under way for the transmission line and excavation had been begun for the power plant. The first locomotives will be received and put into service early in 1925, and the electrification should be in complete operation by the autumn of that year.

Supreme Court Decides Interstate Drainage Suit

Minnesota Not Responsible for Flooding Dakota Stream—States Must Plan Remedial Measures Alone or Together

THE PROTRACTED Dakota-Minnesota interstate drainage suit, in which the former state alleged damage to its citizens by flood waters from Minnesota, and which has been before the United States Supreme Court for some years, has ended in a decision in favor of Minnesota as the defendant. As far as relief by injunction is concerned, the court's conclusion was that the state of Minnesota is not responsible for the floods of which complaint is made, and that the opinions and suggestions of the engineers must be left for the consideration of the two states in any effort by either or both of them to remedy existing conditions. In regard to a decree for payment for damage to farms in the Bois de Sioux valley by the floods of 1915 and 1916, this was denied for lack of jurisdiction.

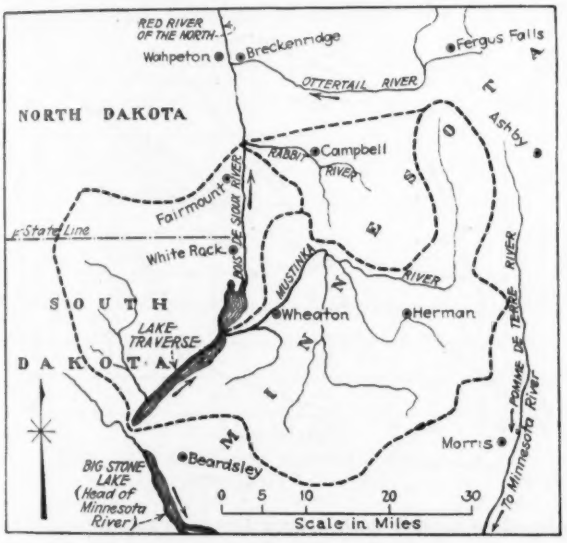
This result was unexpected, since the question of jurisdiction was argued in January, 1921, and in April of that year the court indicated that it had virtually concluded that Minnesota was responsible, as the court restored the case to the docket and called for further evidence in regard to possible remedial measures, such as dredging the Bois de Sioux River or converting Lake Traverse into a storage reservoir. This suit was primarily on account of the injuries to private citizens of the complainant state, comparatively little injury being sustained by the property of the state and other public property, such as highways. Under the eleventh amendment to the Constitution, the federal courts have no authority in suits prosecuted against one state by citizens of another state, and the Supreme Court now concludes that North Dakota cannot collect damages for individuals, although it could secure relief by injunction if it should prove its case.

Engineering Questions Involved—Engineering questions in which there are few legal precedents were the basis of this suit. The complaint of North Dakota (with South Dakota also interested) was to the effect that the construction of an extensive drainage ditch system in Minnesota has added greatly to the flood season flow of the Mustinka River, and has thereby placed an excess burden upon Lake Traverse (into which it flows) and also upon the lake's outlet stream, the Bois de Sioux River. As a result, it was claimed this stream was caused to overflow repeatedly, to the injury of agricultural lands in Dakota.

The area in question surrounds the point where Minnesota, South Dakota and North Dakota meet, as shown by the accompanying map. The Mustinka River has a drainage area of about 800 sq.mi., and discharges into Lake Traverse on the boundary line between South Dakota and Minnesota. The area of this lake at different stages is from 20 to 30 sq.mi. From Lake Traverse the flow is north through the Bois de Sioux River, which forms the boundary line between North Dakota and Minnesota for twenty-three miles, until it joins the Ottertail River at Breckenridge, Minn., to form the Red River of the North (see *Engineering News-Record*, July 29, 1920, p. 225, and July 26, 1923, p. 133). The drainage areas are about

2,080 sq.mi. for the Ottertail River and 1,740 sq.mi. for the Bois de Sioux River at the outlet of Lake Traverse, but the channel of the latter river in its upper reaches, near the lake, is only a few rods wide and a few feet deep below the level of the prairie. In the summer of 1915 and the spring of 1916, Lake Traverse rose and the outlet river overflowed at such time in the agricultural season as to ruin the crops, water standing for weeks upon a strip of land from one to three miles wide along the upper part of the Bois de Sioux River and narrower areas along Lake Traverse. Injury was alleged to about 15,000 acres in Dakota, and damages of about $1,000,000 were claimed.

This flooding was attributed by the complainants to ditching work which includes enlargement and straightening of the still inadequate channel of the Mustinka River for about twenty miles, which was done (mostly in 1912 to 1915) by the state of Minnesota under state



TERRITORY INVOLVED IN DRAINAGE SUIT
Dotted lines indicate drainage areas of the Mustinka, Bois de Sioux and Rabbit rivers.

appropriation, and also a system of large ditches covering the Mustinka valley and built by counties or districts for which the state ditches form outlets. Incidentally, it is stated that in 1922 there was flooding in the Mustinka, but not in the Bois de Sioux. It was claimed that the effect of these ditches is to carry from the Mustinka drainage area an annual runoff materially larger than would have occurred before drainage construction, because the opportunity for evaporation loss was then greater; and that this runoff now occurs in sudden maximum peaks instead of more slowly as would be the case if the water were delayed on the flat Mustinka lands by lack of ditches and discharge channels.

No Long-Time Flood Records Available—The average annual rainfall in the Mustinka valley is 23.8 in. There are no long-period river-gaging station records in this valley, but from such short records as are available and from long records on adjoining streams it was stated that the natural average annual runoff was probably somewhat less than two inches. The maximum discharge of the Mustinka in 1916 was estimated by the engineers for Dakota as about 3,500 sec.-ft., with an average for the maximum ten days of 2,500 sec.-ft. From study of the channel capacities before improvement by dredging and ditching, and assuming a detaining basin designated as the "delta zone," these engineers for the complainant state gave the opinion that without the ditching only about half of this flow could have reached the outlet in ten days, the remainder (except for evaporation and other losses) reaching the outlet after a delay of several days or weeks. They claimed that under such original conditions the whole or a great part of the injurious flooding on the Bois de Sioux valley lands would not have taken place, but under present conditions repetition of the flooding is probable at any time of more than average precipitation.

These estimates of flow were minimized by the engineers for the defendant state, who denied that the ditching added materially to the volume or accelerated its arrival injuriously, largely because there never was a real detaining basin in the "delta zone." They placed almost the entire blame for the flooding upon the excessive and concentrated rainfall of the three years 1914, 1915 and 1916. To this the complainants replied that whatever may be assumed would occur under natural conditions, artificially created conditions have been superimposed upon the former and have necessarily increased the floods, and will continue to do so until suitable remedy is employed.

This position was challenged by the engineers for Minnesota, who presented data which indicated that there had been little increase or acceleration of flow in the Mustinka River as the result of drainage. They showed also that even if there had been material acceleration of flow into Lake Traverse the resulting lake level and flood conditions would have been decreased, so long as the total volume of inflow remained the same, because the more rapid the inflow the greater the opportunity for outflow during the period of rising. As to these issues the court concluded that "the experts called by Minnesota seemed to use more specific and accurately ascertained data for their estimates, and this circumstance, as well as the more satisfactory reasons given, lead us to think their conclusions more to be depended upon."

The Dakota-Minnesota case was initiated early in 1917. During the initial engineering investigation and court hearings, North Dakota was represented by George A. Ralph (who had been state drainage engineer of Minnesota from 1900 to 1913), Francis C. Shenehon, consulting hydraulic engineer, and E. F. Chandler, professor of civil engineering at the University of North Dakota. Minnesota was represented at that time by Frederic Bass, professor of municipal and sanitary engineering at the University of Minnesota, and M. Sardeson, geologist. After the court restored the case to the docket in 1921, North Dakota was represented by Mr. Shenehon and Mr. Chandler, but the state of Minnesota engaged Adolph F. Meyer and Arthur E. Morgan, consulting hydraulic engineers, in addition to Mr. Bass. The decision was rendered in last December.

Canadian National's Financial Status

The annual report of the Canadian National Rys. shows a deficit for 1923, including fixed charges, of $52,779,000 as compared with $58,897,000 in 1922. Gross operating revenue was $263,554,000 as compared with $241,686,000 for the preceding year and gross operating expenses were $242,367,000 as against $237,692,000. The total surplus before fixed charges was given as $13,365,000 as compared with only $1,500,000 for 1923.

Grading Aggregates by Fineness Modulus Method

Abrams Theory Supplemented by Charts for Three Sizes of Aggregates—Scientific Method of Proportioning Made Easy

By C. D. Willett
Assistant Bridge Engineer, California Highway Commission, Sacramento, Calif.

According to the Abrams theory of proportioning concrete, variation in the grading of the aggregates affects the strength of the concrete made with a given amount of cement merely because it affects the amount of water that is required to produce a workable consistency. If the proportion of cement is varied to maintain a constant ratio of cement to water any reasonable grading of aggregates can be made to yield a concrete of approximately any desired strength. An arbitrary quantity known as the fineness modulus is determined by sieve analysis of the aggregates and this quantity together with the maximum size of the aggregates determines the amount of cement to be used.

The strength of the concrete made from the cement and aggregate in the determined proportions will depend upon the amount of water used. Tables and charts based upon experimental data supply the means for the practical application of the theory.

The writer has supplemented this theory with charts showing a uniform grading. This grading has been so worked out that it is readily secured from local deposits of material throughout California. The combination of sand and gravel or crushed rock is shown by the heavy diagonal line. In working up the charts the writer was chiefly interested in keeping down the percentage of fine materials which tend to retain the water and invariably cause the use of additional water on the job.

These charts have been so arranged that the application of the Abrams theory may be used when the materials are graded on standard screens; the heavy dotted lines indicate the Tyler screens. By plotting the gradings of the standard screens the gradings of the Tyler screen may be taken from the chart which will give a very close check on the fineness modulus of the material to be used.

These charts have been worked out for a mix approximating a 1:2:4 concrete, which gives an actual mix of 1 part cement to about 5.2 mixed aggregate, the aggregate quantity varying according to material. The chart for reinforced-concrete bridge work has a maximum aggregate of 1½ in. It is so graded to give a fineness modulus of 5.52, which it has been found will give a workable concrete with most materials. With some materials it has been found that an excellent working concrete could be obtained with the gradings below the heavy line, giving a fineness modulus of 5.6.

The third chart is for a maximum aggregate of 2½ in. This chart is so graded to give a fineness modulus of 6.22. This gives a workable mix for mass or pavement concrete where a concrete approximating a 1:2:4 is specified.

With a good quality cement and aggregate the concrete made according to charts for reinforced concrete, giving a workable mix, will produce a compressive strength of 2,500 lb. per square inch in 28 days. Concrete mixed according to the third chart with a 2½-in. maximum aggregate will produce a compressive strength of about 3,500 lb. per square inch using a dry mix which can be used in pavement concrete.

Illustrations—To illustrate an application of this method under field conditions there is no better example than the results obtained by the writer as superintendent of construction on the new California State Printing Office, completed over a year ago. The building was of a heavy reinforced-concrete construction containing about 3,600 cu.yd. of concrete.

In pouring the footings the writer was compelled to be a little lax in his requirements as to the methods used in mixing and placing as it was necessary to break men in on a method of placing concrete unknown to any of them, with the result that the writer was termed a theoretical fool. In compelling the men to comply with orders it was necessary to fire the mixerman for the first four consecutive days of pouring, because they were determined that the concrete should have plenty of water. They added water at every opportunity. After trying out four experienced mixermen who absolutely

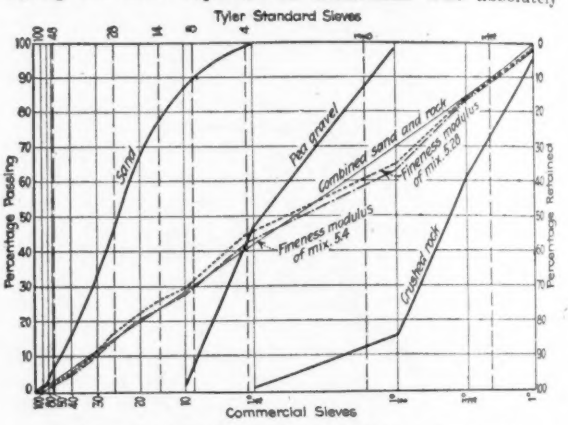


FIG. 1—CHART SHOWING AGGREGATE GRADING FOR 1:2:4 CONCRETE

refused to comply with orders the only solution was to take a man who had no experience in concrete work and break him in on the mixer. This was a success and this man remained on the mixer throughout the job. The test samples on this job were not taken as per standard specifications for test samples. The cylinders were taken in 6x9-in. tin cans with lids which were placed on at the time the cylinders were made and not removed. The cylinders remained on the job for about a week and the average temperature was about 50 deg. F. Under such curing conditions the 6x9-in. cylinder should compare favorably with the 6x12-in. cylinder properly cured. There was an average of two cylinders taken each day of pouring. The aggregate used through the job was an average graded sand with a fineness modulus of 2.80 and an intermediate sandstone gravel a little soft, graded from ¼ to ¾ in., and a very hard crushed stone with a maximum size of 1 in. These were mixed as shown on the chart to give a fineness modulus of 5.40. This grading was mixed to approximate a 1:2:4 mix for the foundations and floor slabs.

The foundations consisting of about 700 cu.yd. were poured fairly dry using a relative consistency of about 1.10 and this gave an average strength of 2,569 lb. per square inch.

The floor slabs consisting of about 1,500 cu.yd. were poured very dry with a relative consistency of about 1.00.

The slabs were poured so dry that there was no visible

water raised to the top after severe tamping. When these floors were stripped there was no steel showing as was expected by many and but very little honeycomb. By this method of pouring shrinkage cracks were eliminated. The average strength obtained in this concrete was 2,869 lb. per square inch. The lighter members of the building such as walls ranging from 4 to 8 in. necessitating at times a 20-ft. drop made it necessary to increase the sand and water.

The fineness modulus of the material was 5.28 and the relative consistency was about 1.25. There was about 400 cu.yd. of this class of material poured, the average strength being 1,891 lb. per square inch.

The columns of the job containing about 225 cu.yd. of concrete were mixed to approximate a 1:1½:3 mix. Owing to the small size of the columns and the close spacing of hooping it was necessary to maintain a fineness modulus of 5.40 and a relative consistency of about 1.20, which gave an average strength of 2,739 lb. per square inch.

The combined mixes are shown in Fig. 1.

A second illustration to show the practicability of the chart is shown in Fig. 2. The broken line indicates a

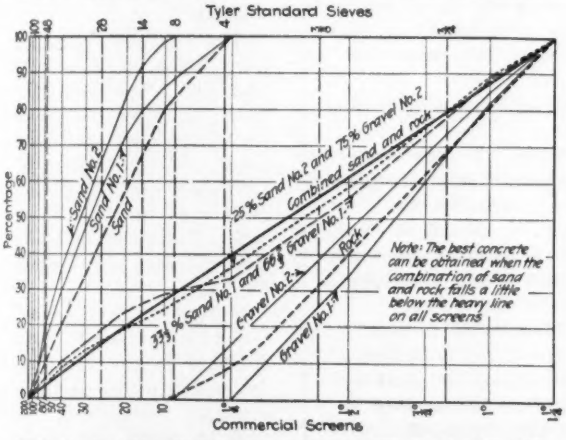

Tyler Standard Sieves
Percentage
Commercial Screens
Sand No.2
Sand No.1
Sand
25% Sand No.1 and 75% Gravel No.2
Combined sand and rock
33⅓% Sand No.1 and 66⅔% Gravel No.1
Gravel No.2
Rock
Gravel No.1
Note: The best concrete can be obtained when the combination of sand and rock falls a little below the heavy line on all screens

FIG. 2—CHART SHOWING AGGREGATE GRADING FOR REMIXED SAND AND GRAVEL

mix in which the sand and gravel were taken from a pit in the Sacramento River and separated on the ¼-in. round screen and remixed in the proportion of 1 part sand to 2 parts rock, to give a fineness modulus of 5.55. The consistency of the mix was such as to give a slump of 3 in. and readily slide on an angle of 30 deg.

To comply with the above requirements the relative consistency required was 1.18. The strength obtained using the standard 6x12-in. cylinder and properly cured was 2,110 lb. per square inch. The dotted line shows the same material in which the sand and gravel were separated on the ¼-in. sieve and remixed in the proportions 25 per cent sand and 75 per cent gravel, giving a fineness modulus of 5.6. The consistency of the mix was such as to meet the requirements of the other.

To comply with the requirements the relative consistency was 1.07 and the strength produced was 3,880 lb. per square inch. These strengths are not comparable owing to the fact that they were put up with different brands of cement, one averaging nearly 1,000 lb. higher than the other.

To secure a constant fineness modulus in construction it has been found advisable to divide the aggregate into three sizes, a fine aggregate or sand, an intermediate and a coarse aggregate. The exact limiting sizes of the different aggregates should be governed according to the maximum size of the aggregate and the character of the gradings of the available materials.

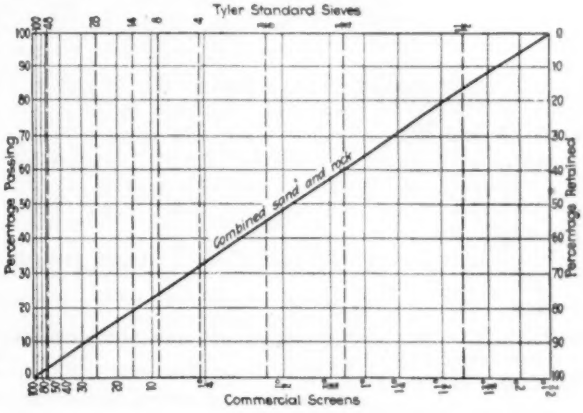


FIG. 3—CHART FOR MASS OR SIDEWALK CONCRETE

The advantages of the chart are:

1. The chart assures a uniform mix with a constant fineness modulus.

2. By the grading thus given quite dry concrete can be worked without difficulty and excess water is easily seen.

3. The chart eliminates all guess as to proportioning of a given mix.

4. Use of the chart makes easy the application of a scientific method to small structures.

5. It enables the use of men as inspectors who are less experienced in the methods of proportioning concrete.

6. It enables the use of the Abrams theory when only standard sieves are available, and gives a picture of the gradings of material.

It is recommended that the lenient grading specifications of the individual aggregates generally specified be replaced by the charts allowing a maximum variation of the combined aggregates of not more than 5 per cent on any one Tyler screen and that the summation of the percentages retained on the 100, 48, 28, 14, 8, 4, ⅜, ¾, 1½ shall not be less than 530 for the 1-in. maximum aggregate chart, 550 for the 1¼-in. maximum aggregate chart and 620 for the 2½-in. maximum aggregate chart, with not more than 2 per cent of the combined aggregates passing the 100 screen.

1923 London's Record Travel Year

Record totals of passengers carried last year by the London transport agencies have been issued. The passengers carried on the subways, buses and trams amounted to 1,630,000,000, which is an increase of 188,000,000 over the total for 1922. The car-miles operated were 217,000,000, an increase of 27,000,000 on the previous year. The subways carried 306 million passengers over 69 million car-miles, the buses 1,134 million over 129 million car-miles, and the trams 190 million over 19 million car-miles. Journeys per capita were 217, an increase of 5 over 1922. The average fare paid per passenger was reduced by 2d. The rolling stock was augmented by 47,000 seats, and the total number of seats now provided is 282,000, an increase of 86,000 since the end of 1919.

Comparative Utility of Concrete Pavement Tests

Results and Analyses of Four Methods of Testing Concrete Pavement—Control Specimens Save Money, and Give Adequate Information—Any Method Gives Satisfactory Crushing Strength Data

By J. G. Bragg
Senior Testing Engineer, State Highway Commission, Trenton, N. J.

EASY determination of the actual quality of concrete in pavements is important. Various methods are employed and they all have value and possess disadvantages. To arrive if possible at some measure of the comparative utility of these methods a series of tests was undertaken last year by the New Jersey State Highway Commission.

All concrete tested was placed on a single project between Aug. 8 and Dec. 4, 1923. The specimens were made about every second work-day and were therefore fairly representative of average weather and working conditions. The section of highway selected for the experiment was located near the laboratory and was built by state forces employing convict labor. Conditions affecting quality of materials and their use were under control at all times. Only bin-tested portland cement was used. The aggregates were river-deposited, glacial sand and gravel, washed and screened to uniform size. Typical analyses are given in Table I. The nominal proportions were 1 : 1½ : 3½, with a specified cement content of 6.72 bags per cubic yard of finished concrete.

Outstanding Conclusions

1. The test methods under consideration are: (a) Control specimens made by the construction inspector; (b) control specimens made by a representative of the laboratory and cured in the field; (c) control specimens made by a representative of the laboratory and cured in the laboratory; and (d) specimens cut from completed pavement by a core drill.

2. Any of the methods is satisfactory for determining crushing strength.

3. By any method, a sufficient number of samples must be taken to obtain a fair average; a fair average is obtained on ten specimens.

4. Conclusions should not be based on less than ten specimens per mile of road or approximately one specimen per 1,000 sq.yd. of finished pavement; a greater number is preferable.

5. It is not practicable to attempt to take a sufficient number of proper-size cores to be tested at 28 days or less for determination of crushing strength; the results are not commensurate in value with the amount of money expended.

6. A considerable saving is accomplished and very satisfactory data are obtained by the use of control specimens in conjunction with the core drill; this is therefore a proper procedure.

The specimens taken by method (a) consisted of 6-in. cubes and no attempt was made to direct or supervise the making or handling other than the transmittal of general laboratory instructions which are given to all construction inspectors. These samples were removed from the mold about 24 hours after casting, stored in water five to seven days and forwarded to the laboratory where they were kept sprinkled until removed from the damp room for testing.

The other three methods (b), (c) and (d) were at all times under the direct supervision of the laboratory and all specimens taken from a designated station were from the same batch of concrete. In each case the selection of the batch depended on its location only, i.e., care was exercised to see that it was placed in such a position that no concrete from other batches became mixed with it in tamping or finishing. After casting the molded specimens, the center of the batch was carefully located and approximately three weeks later the core was drilled. The molded cylindrical specimens (methods b and c) were 6 in. in diameter and 9 in. in height. These dimensions were selected because the 6-in. diameter cores capped for testing averaged 9 in. in height.

Specimens taken by method (b) were removed from the molds 24 to 48 hours after casting and buried in earth alongside the pavement where they remained until the time of drilling the core. They were then brought to the laboratory and kept with the cores until tested.

Specimens taken by method (c) were removed from the molds 24 to 48 hours after casting and brought to the laboratory where they were kept sprinkled until removed from the damp room for testing.

The results obtained are shown in Tables II and III. The strengths have all been corrected for heights of specimens equal to two diameters, the corrections applying mainly to the 6-in. cubes where the actual breaking load obtained was divided by 1.4.

Variation in Strength—Attention is first directed to the variation in strength between individual specimens. It will be noted that while there is fairly close agreement between molded specimens from the same batch there are wider variations between these specimens and corresponding cores. This is to be expected since the molded specimens were always cast in identical forms, tamped to the same degree and the curing conditions during the early curing period (2 to 3 weeks) were quite uniform. On the other hand the concrete from which the cores were cut was subject to such indeterminate variables as: (1) subgrades having different affinities for moisture, (2) different degrees of consolidation, (3) variations of temperature and (4) variations in curing due to improper sprinkling, rains, hot suns, wind, etc.

It will be noted also that all methods indicate a wide variation in the strength of the concrete throughout the section. There are, of course, many contributing factors to cause such fluctuation in strength among which are changes in the gradation of fine and coarse aggregate and insufficient mixing of the concrete. In this case, however, in order of their importance, the differences in strength are attributed to variations in proportioning

TABLE I—SCREEN ANALYSIS OF FINE AND COARSE AGGREGATES

Fine Aggregate			Coarse Aggregate	
Passing	Retained	Per Cent	Screen	Per Cent Passing
½ in.	¼ in.	3.8	2½ in.	100
¼ in.	10 mesh	16.46	1½ in.	46
10 mesh	20 mesh	13.95	¾ in.	1.5
20 mesh	30 mesh	22.68	¼ in.	.2
30 mesh	40 mesh	18.59		
40 mesh	50 mesh	15.99		
50 mesh	80 mesh	8.44		
80 mesh	200 mesh	3.25		
200 mesh	.....	.65		
Elutriation		1.9		

Note—Mortar tests at seven days showed a strength of 111.8 per cent of standard Ottawa sand, and at 28 days of 109.1 per cent.

materials and amount of water used. The construction records show that the cement content varied from a 3.2 per cent shortage to 13 per cent excess for a day's bags per cubic yard and required depth of 8 in. The core drill records show also that the pavement varies in depth from 7.5 in. to 10.4 in. with a general average of 8.5 in. The following example demonstrates clearly how the strength may be affected due to the above variable. Three cores cut from the first day's work averaged 8.1 in. in depth, showing a strength of 3,300 lb. per square inch. Two cores cut at a later date averaged 8.8 in. in depth, showing a strength of 1,923 lb. per square inch. The quality of materials and consistency were known to be comparable on both days. The construction records show however that 13 per cent excess cement was used in the 8.1-in. slab as compared with a 1 per cent shortage in the 8.8-in. pavement. The percentages of excess and shortage were in both cases based on the specified depth of 8 in. It is true also that this difference in strength could have been further augmented by an excessive amount of mixing water.

This investigation shows quite conclusively that when all the variables which affect the strength of portland cement concrete are given proper consideration, no one of the methods discussed herein has any great advantage over the others so far as determination of quality is concerned. There is, however, a great difference in the cost of obtaining so-called control specimens and core-drill specimens and this matter is discussed in a later paragraph. The figures obtained supported also the point upon which most investigators are agreed, that such data are of small value unless, for any one condition, a sufficient number of samples are taken to obtain a fair average. The averages given at the bottom of Table III indicate that where 10 or more specimens are averaged the several methods agree within practical limits.

The question of considering all variables and obtaining fair averages is particularly stressed because of the fact that since the advent of the core drill certain

TABLE II—COMPRESSIVE STRENGTH OF 6-IN. CONTROL CUBES TESTED AT 28 DAYS

Lb. per Sq.In.	Lb. per Sq.In.	Lb. per Sq.In.	Lb. per Sq.In
2,558	1,659	2,295	2,451
2,744	2,558	2,141	2,141
2,633	2,121	2,360	2,021
2,507	2,446	2,107	2,618
2,812	2,144	2,080	2,384
2,617	2,337	2,449	2,205
2,439	2,806	2,806	2,621
2,347	2,586	2,189	2,726
2,549	2,357	2,693	1,859
2,018	2,252	3,665	2,635
	2,466	2,545	2,957
General average			2,438
High 3rd average			2,765
Medium 3rd average			2,442
Low 3rd average			2,082

Note: All results are corrected for height equal to two diameters of specimen.

articles have been published in which some rather absurd conclusions were drawn from results which could not possibly be comparable. In most instances either the age of the tested concrete varied throughout the experiment, no corrections were made for the form of the specimen used or too many projects were included with an insufficient number of specimens taken on each project. One writer remarks the astonishing fact that cores cut from the finished pavement showed much higher strengths than did the control specimens made when the concrete was placed. In view of the fact that his control specimens were 6x12-in. cylinders tested at 28 days, some of the 6-in. cores cut were two years old with a height of 6 to 8 in., and no correction made on the results for height to diameter ratio, it would indeed have been remarkable had he not experienced higher strengths in the field-cut cores.

Core-Drill Specimens—Drilling 6-in. diameter cores is a costly operation as compared with either of the

TABLE III—COMPARATIVE COMPRESSIVE STRENGTH AT 28 DAYS OF FIELD CONCRETE MOLDED AND DRILLED SPECIMENS

	Compressive Strength Lb. per Square Inch		
Date Concrete Placed	Method (b) Molded Field Cured	Method (c) Molded Lab. Cured	Method (d) Drilled Core
Aug. 8	2,220	2,414	1,769
Aug. 13	2,712	2,796	2,541
Aug. 15	2,272	2,252	2,218
Aug. 17	2,549	2,744	3,054
Aug. 20	3,105	2,682	3,235
Aug. 23	3,352	3,279	3,225
Aug. 25	2,579	2,341	2,619
Sept. 1	2,467	2,229	3,275
Sept. 5	2,546	2,594	2,759
Sept. 7	2,579	2,311	3,172
Sept. 10	1,771	1,749	1,852
Sept. 10	2,650	2,622	2,501
Sept. 11	2,010	2,022	1,945
Sept. 17	2,139	1,933	1,764
Sept. 20	2,755	3,241	2,102
Sept. 26	2,414	2,634	2,231
Sept. 28	2,739	3,171	2,506
Oct. 1	2,206	1,988	1,799
Oct. 3	2,467	2,660	2,779
Oct. 5	2,462	2,647	3,835
Oct. 11	2,380	2,431	1,745
Oct. 13	2,857	2,945	3,102
Oct. 17	2,344	2,054	1,919
Oct. 19	2,475	2,582	2,637
Nov. 3	3,625	4,245	2,577
Nov. 9	1,294	1,678	2,407
Nov. 9	1,922	1,596	2,496
Nov. 13	2,706	2,483	3,320
Nov. 16	3,587	3,206	3,053
Nov. 19	1,970	2,464	2,209
Nov. 26	3,425	3,060	1,821
Dec. 4	3,199	2,351	2,682
General average	2,559	2,544	2,530
High 3rd average	3,106	3,094	3,164
Medium 3rd average	2,506	2,506	2,474
Low 3rd average	2,015	1,981	1,893
Average of 1st ten	2,648	2,564	2,787
Average of 2nd ten	2,361	2,467	2,331
Average of 3d ten	2,516	2,568	2,527

other methods. This is true when no attempt is made to test the cores at a predetermined age. If they are to be tested at 28 days, the accepted practice for control specimens, the cost mounts up rapidly because of the cutting time lost in moving from one job to the next. In the light of past experience on our own roads it is conservatively estimated that one core-drill crew, cutting ten cores per mile of road, could keep up with a maximum of eight mixers. To do this the drill must visit each job every tenth workday with the result that it is operating only two days in every three. Forty projects would require five core drills operating continuously and with five needed in the field it would be necesary to have on hand a sixth drill for emergency use. Control specimens are obtained at a very small fractional cost as compared with core drill expenditures.

This is not in any sense a condemnation of the core drill, which serves a purpose not covered by control specimens. It tells us whether the pavement has the required depth and density, also whether the reinforcing metal is properly spaced and other like information on which may be based future design and construction

practice. For this purpose, however, and when they are not to be tested for strength, smaller than 6-in. diameter cores may be cut. The small cores are cut much faster than the larger ones with less depreciation of equipment and a corresponding reduction in cost at all points.

It would seem, therefore, that casting control specimens during construction in combination with the core drill operations on finished pavements is a proper procedure. Certainly it produces accurate records with a well worth-while reduction in cost to the department. By this procedure two drill crews will adequately handle the 40 projects mentioned above. Incidentally the annoyance to the contractor of having a core drill on the job every two weeks is no small item and this is particularly true where the pavement is being constructed in half-widths with traffic maintained at all times or with the contractor's trucks traveling over the completed portion of the pavement. Such annoyances must be paid for and certainly after experience with the first contract it will not be the contractor who pays.

So far the strongest objection to the use of control specimens when made by the construction inspector is the accusation that he may be dishonest in making them. He may choose only the best concrete from which to make his specimens, he may tamp the specimen too much or he may even add some cement. This array of "he mays" fills us with dismay when we contemplate all the other things such an inspector may do. The answer to this seems to take the form of another question. If the inspector's integrity is such that he cannot be trusted to honestly fulfill this small part of his duties, should he be intrusted with his other duties?

If for any reason, however, it is impracticable to have the control specimens made by the construction inspector, they may be made by representatives of the laboratory each of whom could cover several jobs. While specimens thus taken would cost more than those taken by the construction inspector, the cost would still be very much less than for drilled specimens.

Wood Floors for Wyoming Bridges

SEVERAL designs of floor have been used in the 75 treated lumber bridges erected in Wyoming by the State Highway Department during the past year. J. F. Seiler, bridge engineer of the department, states in *Wood Preserving News* that the best service has been given by a floor of 4 x 12-in. planks grooved on the edges for splines. By this design concentrated loads are distributed over two or more planks, and consequently the stringer spacing may be increased. The solid planking across the stringers is held in place at the corners by 4 x 4-in. strips. These also serve as stop-diaphragms in case of fire.

In order to keep the floor plank from working loose the felloe-guards are bolted down directly to the floor with long bolts extending through the outer stringer. Two 7-in. spikes fasten the planks to each intermediate stringer.

All floors have wearing surfaces of from ½ to ¾ in. in thickness with an excess of small gravel or stone chips and sandy loam spread on top to take up any bleeding or irregularities in the surface. The grooved plank is surfaced on one side and two edges, and is laid with rough side up and ½-in. cracks between the planks. The asphalt runs down the crack to the spline, which has a tight up and down fit and a clearance at the sides for expansion.

Union Pacific Loop Puts Boise, Idaho, on Main Line

Extending Branch Back to Main Line Gives Heavier Grades and Curvature, but Capital City Is Now on Through Route

WITH THE completion of a 28-mile railway link now under construction by the Oregon Short Line R.R. (Union Pacific System) the city of Boise, Idaho, will be placed on a through main line instead of being served only by the Nampa-Boise branch of the above railway. Since Boise is the capital and the largest city in the state, there has long been a desire on the part of the citizens for through main-line service and it is probable that this desire will be gratified before the end of 1924. The improvement consists in extending the present branch to reach the main line again at Orchard, thus forming a loop over which some of the through trains will be routed. The situation is explained by the plan and profile, Fig. 2, while

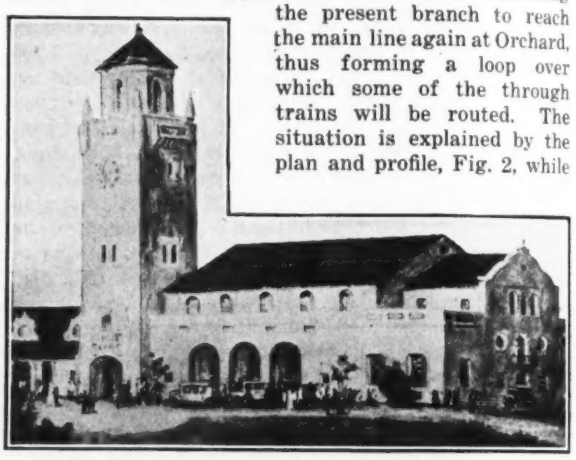


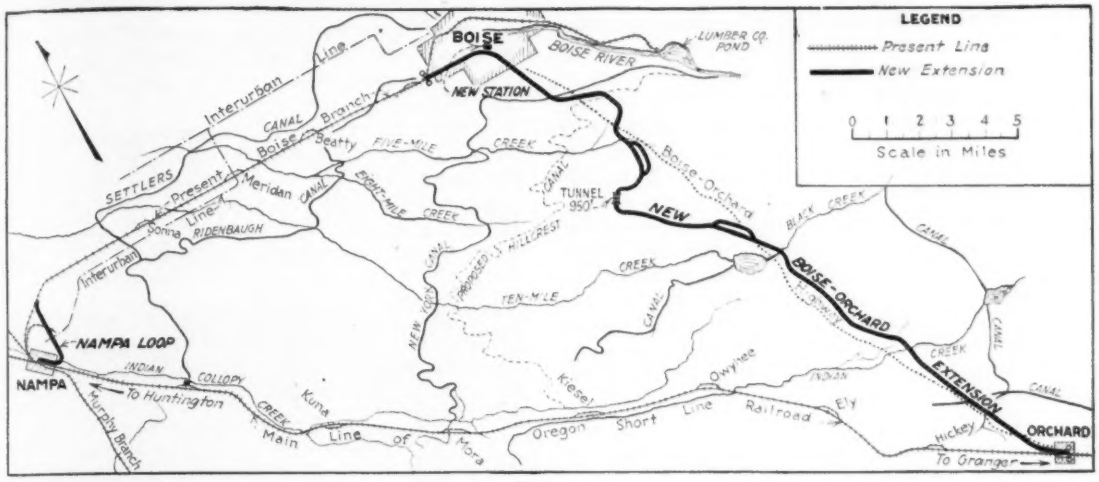
FIG. 1—NEW STATION FOR BOISE, IDAHO

the characteristics of the two alternative routes are shown in the accompanying table.

It will be seen that the new line through Boise will be about ten miles longer than the present line between Orchard and Nampa and will involve nearly ten miles of 1 per cent grade eastbound. For this reason it is not anticipated that the new line will be serviceable for through freight traffic, except during the movement of heavy business in fall and winter months, when through westbound freight trains might be moved between Orchard and Nampa by this route, thus avoiding the congestion on single track where the grade between Orchard and Nampa is the ruling grade for eastbound business on the district between Glenns Ferry and Nampa. Grade and line improvements at Glenns Ferry were described in *Engineering News-Record*, Sept. 6, 1923, p. 388.

At Boise, the new route does not reach the lower level of the town but is located on the first bench above the Boise River south of the main portion of the town. A new station of handsome design, Fig. 1, will be erected on the extension of Seventh St., which is the axis of the center of the capitol building. It is expected that eventually the city of Boise will construct a wide boulevard between the capitol and the new station. In fact, the city is arranging already to establish a park on a small area of the bluff in front of the station site.

In order to avoid backing into the station at Nampa,



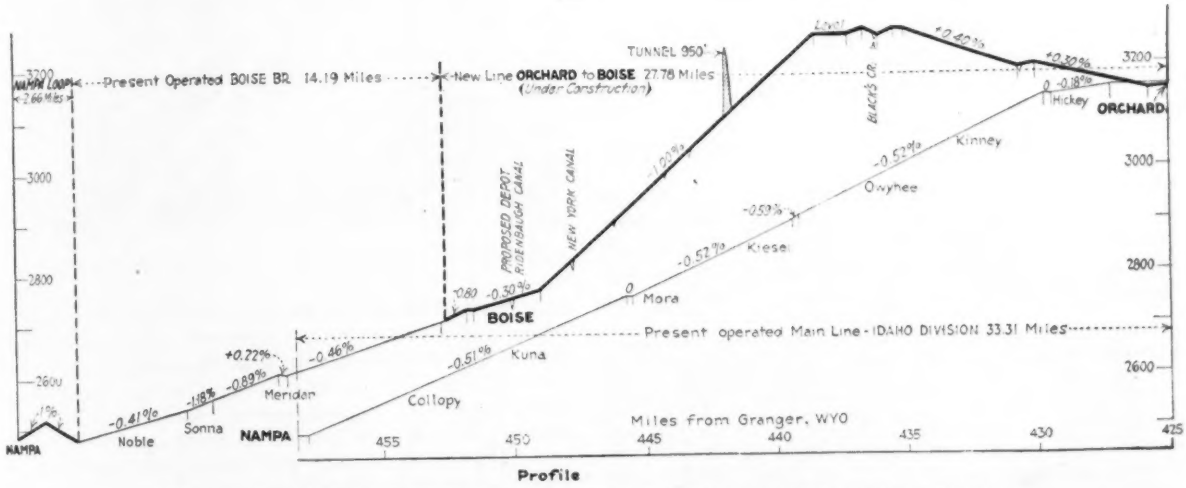


FIG. 2—MAIN LINE AND BOISE LOOP: OREGON SHORT LINE (UNION PACIFIC RY.)

a loop is being constructed between this station site and a point two miles from town on the existing Nampa-Boise Branch. As this loop line is partially in the city, grade separations over three highways leading into the town are being made. A new station at Nampa is part of the program, as well as the reconstruction of the main line through the station grounds in order better to serve the new station and also to relieve congested conditions in the yards. Railway business at Nampa, in the center of a large fruit and agricultural district, has increased rapidly, and as this city is at the end of a number of branches and is equipped with car icing and other facilities, it is the starting point of a considerable number of trains of perishable freight during the season. At Orchard there will be a new coaling station of steel construction with 150 tons storage capacity.

NEW BOISE LINE: OREGON SHORT LINE R.R.

Orchard to Nampa	By Present Main Line	By New Boise Loop
Present main line	33.31 miles	.....
New line to Boise	.....	27.78 miles
Present Boise branch	.....	14.19 miles
New Nampa loop	.....	2.66 miles
Total miles	33.31 miles	44.63 miles
Maximum curvature	3° 00′	4° 00′
Total curvature, degrees	352° 40′	1,194° 43′
Total curvature, length	6.38 miles	9.85 miles
Average curvature per mile	10° 45′	26° 46′
Maximum grade, ascending west	None	1.00%
Maximum grade, descending west	0.59%	1.00%
Total ascents, west	None	177 ft.
Total descents, west	664 ft.	843 ft.
Bridges and trestles, length	(8) 572 lin.ft.	(26) 1,561 lin. ft.
Tunnels	None	(1) 950 lin. ft.

Track on both the new line and the present branch will have 90-lb. rails of the A.R.E.A. section, in 33-ft. lengths spliced with four-bolt continuous joints 27 in. long. Zinc-treated pine ties will be spaced eighteen to a rail length, in gravel ballast 9 in. deep under the ties. The entire line will be equipped with automatic block signals. Three passing tracks 4,000 ft. long and a loading track 600 ft. long will be built between Boise and Orchard. For these turnouts, 16-ft. switch rails and No. 10 frogs will be used.

Construction of the Orchard-Boise line was commenced in November, 1923. Grading forces were established in eight camps and a large part of the light work was done during the fall and early winter, as the climate was favorable, so that the larger part of this light work was accomplished before ground was frozen. Heavier work has been carried on during the winter. A 950-ft. tunnel, through cemented gravel, was started in November, 1923, and is expected to be completed in October, 1924.

Grading on the larger part of the mileage is being handled by teams, but steam shovels are used for three miles of heavy excavation about ten miles east of Boise. The bridge work is generally light and the largest steel structure is a 102-ft. through plate girder span over an irrigation canal. Timber trestles are used mainly for smaller structures and there is a large amount of reinforced-concrete pipe for culverts.

The general contract for grading was awarded to the Utah Construction Co., Salt Lake City, but track-laying and steel bridge erection are handled by the railway company's forces. All this work is under the direction of R. L. Huntley, chief engineer, Union Pacific System, and W. R. Armstrong, assistant chief engineer, Oregon Short Line R.R., with L. W. Althof in resident charge.

High-Lift Hydraulic Dredge Has Toothed Cutter Head

Designed for Deep Digging and Long Discharge In Welland River Canalization of Chippawa Power Canal

By V. J. Milkowski
Engineer in Charge, Dredge Department, Morris Machine Works, Baldwinsville, N. Y.

DEEPENING the lower stretch of the Welland River, which forms a part of the Queenstown-Chippawa power canal, calls for rather deep excavation in a hard tenacious clay and long-distance disposal of the excavated material. It was decided by the contractor to have built a hydraulic dredge designed for the special conditions. As ample current was available all along the work, electric power was chosen. The other conditions which governed design were: Depth of excavation 38 ft. below water surface; discharge lift 30 ft. above water surface; length of discharge line 5,000 ft.; a specified 20-in. pump to handle solids up to 14-in. diameter, and 350 cu.yd. per hour capacity.

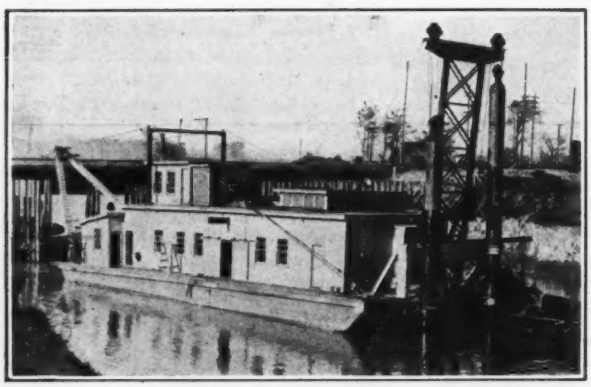


FIG. 1—DREDGE OPERATING IN WELLAND RIVER NEAR CANAL ENTRANCE

The dredge as designed is shown by Fig. 1. An available hull 100x36 ft. and 8 ft. 9 in. deep was converted in order to cut down the time of building the dredge. Particular interest, therefore, lies in the equipment—specifically in the pumping unit and in the cutter design. Mention may also be made of the high pressure flexible coupling for the discharge line, of the speed hoist and of the collateral electrical outfit, but other equipment may be passed with the remark that it was ample and adequate both in design and in character.

To meet the conditions, the pressure which the pump had to develop was greater than had been previously attempted with a single unit. After study, choice was made of a high-duty pump, Fig. 2, directly connected, through a flexible coupling, to a 1,500-hp. slip-ring motor, with a full-load speed of 485 r.p.m., and with

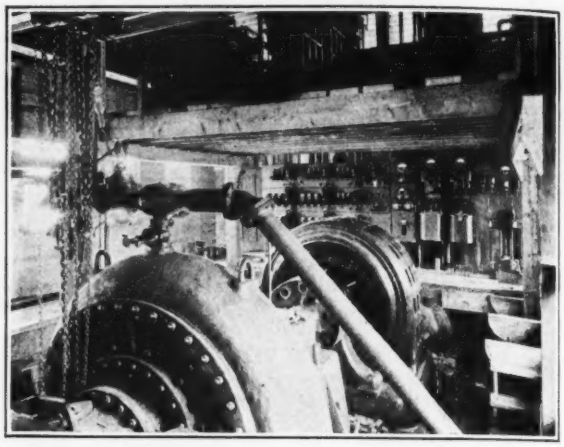


FIG. 2—ELECTRIC PUMPING UNIT WITH CONTROL BOARD

secondary control for 25 per cent reduction. Both power and speed were in excess of anything that had been attempted before with a 20-in. pump. Briefly the reasons for selecting the high speed pump, with a small impeller, in preference to a slower unit, were to reduce the cost and weight of pump and motor, and especially of the pump parts and also to secure a mechanically more efficient machine.

The original pump was entirely of high-carbon steel, but on subsequent replacements the shell and impeller and the throat piece were made of manganese steel. The alloy resisted better the abrasion in pumping sand and gravel. To cut down vibration and shocks—intensified by the high speed and high pressure—the pump, bearings and motor were mounted on a single cast-iron sub-base of heavy box construction.

In detail the pump shell is of the volute type 3½ in. thick at the periphery and is fitted with discs on both sides, these having renewable steel plate liners 1½ in. thick. The impeller is of the inclosed type, hydraulically balanced for end thrust, and is fitted on the shaft by means of a taper-fit nut and key. The shaft is forged steel 7 in. in diameter and has five thrust collars forged integrally with the shaft. On the suction side the disc is fitted with a throat or suction piece, projecting into the impeller. The disc on the hub side is fitted with a water seal which is supplied with clear water from a service pump. The shaft is carried on two steady-bearings of the ring-oiling type, located between the pump and the motor and entirely separate from the pump. A five-collar marine thrust bearing entirely inclosed is located between these two steady-bearings.

With the pumping unit worked out as described, the next item to receive special consideration was the

FIG. 3—SPECIAL CUTTER FOR HARD CLAY

cutter, as it was expected that the usual type would become clogged in the clay which was unusually sticky and firm and could be cut into regular shavings. The cutter adopted is shown by Fig. 3. It was designed with special consideration to the correct pitch and curvature of the blades so that the material would not adhere to the blades, but would be carried through with the water to the suction intake. Better to resist the wear and to provide for renewals, the blades were made with manganese steel cutting edges, which are riveted on and which can be renewed when worn out. To break up the shavings of clay in order to prevent their being rolled up into large balls, these cutting edges were made with teeth at intervals.

The cutter has six blades and is approximately 68 in. in diameter and 60 in. long. It has operated successfully in all kinds of material, and no tendency to clog has been noticed. The cutting edges have been renewed very readily when worn out, and the cutter restored to its original efficiency. The ladder is 65 ft. long from the center line of the trunnion supports to the end of the cutter. The suction pipe enters through the starboard trunnion, while the cutter drive shaft passes through the port trunnion. The motor is 150-hp., 480-r.p.m. full-load speed, with secondary control for 33 per cent speed reduction. The gear reduction is about 37 to 1, giving the cutter a speed of about 13 r.p.m., with the motor operating at full speed.

The hoisting equipment was designed to handle the dredge rapidly on account of the comparatively small and light hull. The hoist was equipped with five drums —one drum being used for raising and lowering the ladder, two other drums for the two forward swinging lines, and the two remaining drums for the operation of the spuds. The hoist is driven by a 60-hp. slip-ring motor, 720 r.p.m. full-load speed with secondary control for 50 per cent speed reduction.

For priming and for general service on board, there are provided two 4-in. double-suction centrifugal pumps directly connected to 30-hp., 1,500-r.p.m. squirrel-cage motors. Only one pump is necessary for this service, the other being provided as a spare.

Controls for the hoist and cutter motors and the levers for the frictions and the brakes on the hoist are located in the pilot house, with gages, and meters from the dredging pump and the main motor so that the operator has complete control of the dredge and a complete check on the operation at all times. The controls for the dredge-pump motor and the service-pump motors are located in the main deckhouse, and are operated from below.

The current is brought on board the dredge by an armored submarine cable, and all of the motors are 3-phase, 25-cycle, 2,300-volt. A small transformer furnishes low voltage current for the lights.

The discharge pipe is carried ashore on pontoons, and a special type of pontoon coupling is employed on account of the high pressure, which makes it impracticable to use the ordinary rubber sleeve pontoon coupling. The coupling adopted is a modification of a ball joint, the ball being held in the socket by a ring forming a part of the socket and a gland at the same time. The joint is made tight by means of a rubber packing ring between the socket and the ring. The coupling can be disconnected very readily by loosening the swivel bolts on the socket, which engage the ring. It is capable of a deflection of 22½ deg. either way, and remains absolutely tight under all conditions of operation.

The conditions under which the dredge has been operated so far have varied so widely and so frequently

FIG. 4—DISCHARGE WHEN EXCAVATING CLAY

that it has been impossible to secure any record of its performance or to ascertain what it is capable of doing under fair conditions. Its performance so far, however, indicates that the dredge can deliver readily even through 7,000 ft. of pipe line with a 10-ft. lift. Observations taken at a time when the dredge was operating in very hard gravel, which had previously been found too hard for another dredge, were as follows:

Depth, ft.	25
Length discharge line, ft.	4,600
Lift above water surface, ft.	10
Pressure head at discharge, ft.	160
Vacuum, in.	16
Speed, r.p.m.	460
Hp. of main motor	1,250
Hp. of cutter motor	125

The performance of this dredge has been so much superior to the performance of the other dredges owned by these contractors that they are now altering their 20-in. steam operated dredge "Shuniah" into an electrically operated dredge, making it as nearly as possible a duplicate of the dredge "Stewart." The contractors are E. O. Leahey & Co., Ltd., of Ottawa, Ont. The dredge was built by the Morris Machine Works, Baldwinsville, N. Y.

Experts Charge Gleno Dam Failure to Masonry Base

Advance Abstract of Official Report on Cause of Disaster—Base Defective in Both Design and Construction

A BOARD of experts appointed immediately after the disastrous failure of the Gleno dam in northern Italy on Dec. 1 last to investigate the cause of the accident has just rendered its report. The full text of this report is not yet available, but its principal findings have been summarized by A. De Martini, of Genoa, who reported the facts of the failure for *Engineering News-Record* (Jan. 31, p. 182).

Two problems were placed before the experts: First, after full investigation of the design, construction and history of the Gleno storage project, to determine the causes of the disaster; second, to consider whether the location of the impounding reservoir on the floor of a glacial basin was proper, with regard to the ability of the underlying strata to hold the water.

Reviewing the history of the dam, the experts state that the original plan for the storage dam contemplated a structure of gravity type. After the construction of the gravity dam was under way, the plan was changed by substituting a multiple-arch design for the gravity design. The lower portion of the gravity dam, already built, was used as a base or platform upon which to build the multiple-arch dam. This base was pierced at about the middle of its length by a large arched opening intended as a drain outlet; a concrete arch closed the upstream end of this opening.

The report states that even during construction of the dam many leaking joints or cracks developed, which increased as time went on. Many strong jets of water spurted through the dam, and before the break the combined flow from these leaks was sufficient to develop a power of 500 kw. in one of the generating units of the power plant below.

Cement mortar should have been used for the masonry base of the dam, but in its place lime mortar was used, of locally manufactured lime, which according to the testimony was none too well made and mixed. The base was constructed on a sound rock foundation, with some attempt to roughen the rock surface but without proper leveling.

No marked weakness existed in the design of the concrete cellular dam built on the masonry base. Even if the ingredients of the concrete had been highly defective and the execution of the work poor, the dam would probably have been able to support its water load; but the gravity dam, that is, the masonry block, was little more than a random heap of stone and lime, poorly bonded, founded and built in chaotic manner, and separated into two parts by the drain opening. Statical analysis and laboratory tests of the materials amply confirmed the view that the base block was in much more dangerous condition than the piers and arches of the superstructure.

No foundation weakness was discovered, and no evidence of earthquake shock (a rumored cause) could be found. In view of these facts the failure of the dam is believed to have occurred in the following manner: The masonry base suddenly gave way at one point; this deprived part of the multiple arch structure above of support, and overstressed the arches to the point of rupturing them in the most highly stressed region. This initial failure caused the destruction of one pilaster and the two adjacent arches, and the impact of this failure and the action of the outpouring water then tore away the rest of the superstructure. This course of events agrees with the account of the watchman at the dam, who reports having seen first a vertical crack in the concrete of the superstructure, followed very shortly by the complete failure of the structure.

The underlying cause of the disaster is established beyond doubt in the insufficient static resistance of the base masonry in the central portion of the dam. Neither the dimensions nor the strength of this masonry was adequate for the forces brought to bear upon it by the thrust of the water upon the dam. The inherent weakness of the base was aggravated by its poor foundations and by the interruption of its continuity by the drain outlet. Final failure was brought about by progressive intensification of this statical inadequacy of the masonry base, up to the moment where the pressures overcame the resistance of the masonry and the resulting settlement so disturbed the equilibrium of the entire structure as to bring about an abrupt failure of the masonry in place of continuous settlement into more compact condition, as often happens in the disintegration of masonry construction.

A fuller account of the experts' findings is expected to be available at a later date.

Water-Borne Disease Epidemic in South Pasadena

Sewer Leakage Enters Low-Pressure Water Pipe Through Holes Ascribed to Sewer Gas Pollution Ratio One to Ten

AN EPIDEMIC of intestinal diseases that occurred recently in South Pasadena, Calif., was attributed to leakage from a sewer which gained access to a low-pressure water pipe through holes believed to have been made by the corrosive action of sewer gas. When the pollution was discovered the water supply being drawn from this particular source was about one-tenth sewage. The corroded low-pressure main through which the contamination came was speedily replaced with a new pipe and a disinfectant was used in the reservoirs. For the month after the water supply was again safe, only eighteen typhoid cases were reported.

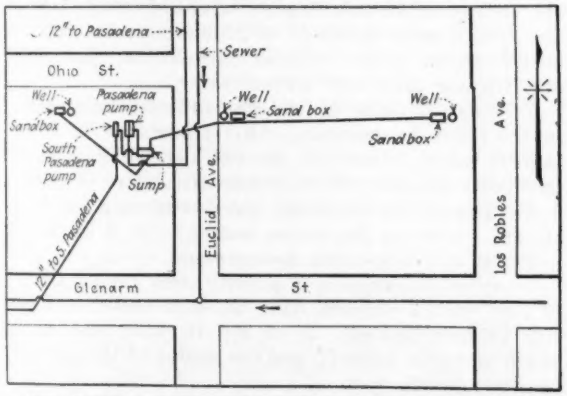
The pollution occurred when, after a shutdown of six weeks, the Ohio St. pumping station in Pasadena which supplies water to South Pasadena was placed in operation at 11 a.m., Saturday, April 26, and was operated intermittently until the Tuesday following at 1:30 p.m. From Sunday night to Wednesday over 200 cases of severe vomiting and diarrhea occurred in the area supplied from this source. In some instances whole families were affected. The complaints were not made until Tuesday and were nearly all telephoned to the city officials of South Pasadena. Being unprepared, the city at first kept no record of the number or location of the complaints, but it was soon apparent that all had their origin in the water supplied by the Ohio St. plant.

South Pasadena has a population of about 15,000 and though not a portion of the city of Pasadena is partially served from the Pasadena water system. About 75 per cent of South Pasadena is supplied from Foothill Reser-

voir (capacity 1,250,000 gal.) and the remaining 25 per cent is served by Grand Ave. reservoir (capacity 900,000 gal.). Foothill reservoir is filled from the Morengo wells augmented by water purchased from Pasadena. The Ohio St. well water is used while the pumps are being operated during the day and the Villa St. reservoir is drawn upon chiefly at night, the ratio being about 60 to 40. The Grand Ave. reservoir is supplied mostly from Ohio St. wells and partly from Villa St. reservoir.

S. B. Morris, chief engineer of the Pasadena Water Department, was informed of the outbreak on Tuesday morning. He found that the sewers in the vicinity of the Ohio St. station and the Morengo station had a stoppage in the 16-in. main outfall sewer about six blocks below the Ohio St. station. After taking a series of samples from the Ohio St. wells, sumps, reservoirs and taps in South Pasadena, he shut down the station at 1:30 p.m. and disinfected the reservoirs by the use of chlorine solution scattered over the surface through cracks in the reservoir roofs. A spare chlorinator was installed Tuesday afternoon and the 12-in. riveted steel pipe line across Euclid Ave. was replaced, the crew working all Tuesday night. Owing to rain falling Wednesday this plant was not used for ten days.

Samples showed the water in the sump at 6 p.m. Tuesday (after pumps had been shut down 4½ hr.) to be

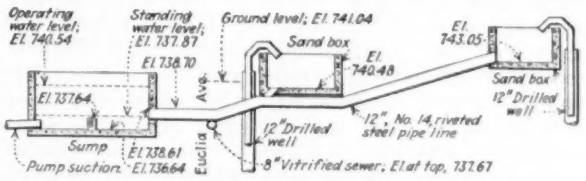


RELATIVE LOCATION OF WELLS AND PIPE LINES

nearly 10 per cent sewage, having a total bacterial count on agar at 37.5 deg. C. of 400,000 per c.c., and a B. coli content, partly confirmed, of at least 100,000 per c.c. During the period of pumping from the sump at the Ohio St. station on Tuesday, the water had a total agar count of 980 per c.c. and B. coli present in two out of three 10-c.c. portions. Both reservoirs showed approximately the same contamination—4,600 and 4,900 total agar count and B. coli present in at least 0.1-c.c. portions. Tap samples had an agar count of about 1,500 per c.c. and at least 10 B. coli per c.c. The well samples were safe, having a total count varying between 0 and 70 per c.c. with no B. coli in 30-c.c. portions. The operator reported that the water in the sump on Saturday morning before operations appeared milky.

Supplying Ohio St. are three 12-in. wells of depth 250 ft., with the water plane 70 ft. below the surface. The water is delivered to collecting sumps or reservoirs adjacent to the wells and flows by gravity or very low pressure to a combined sand catcher and sump near the pumping station. It is pumped from there into the mains leading to Foothill reservoir and to Grand Ave. reservoir. The two wells on the lot across the street discharge into a single gravity line to the station. This conduit was a 12-in. No. 14 riveted steel pipe line installed about 15 years ago. Where the pipe line crossed directly over an 8-in. vitrified sewer in the street the steel pipe was badly corroded and had about a dozen holes varying from the size of a pinhead to ¼ in. in diameter; sewer gas was the cause of these.

The soil at this point is rather heavy, similar to adobe. About 25 ft. away from the crossing the macadam pavement showed a distinct raveling but no wet spot in the street. The sewer in this vicinity had a cover of about



PROFILE SHOWING SEWER AND RELATIVE ELEVATIONS

2.75 ft. and when exposed revealed a longitudinal crack in the crown thought to have been caused by a steam roller a year or so previous. It was further ascertained that during the two weeks just before this outbreak, and on previous occasions, the main sewer system into which the South Euclid line emptied had overflowed through an emergency underground overflow into a covered storm-water conduit three blocks away. These overflows were believed to have been sufficient to back the sewage up about a foot higher than the top of the gravity water pipe in the street. Mr. Morris found that the stoppage was caused by some 2 x 4-in. timber and a cast-iron dirt pan that had dropped from its seat and lodged across the sewer mouth at the manhole on a 16-in. outfall sîx blocks below the Ohio St. station. In the next manhole, another dirt pan was lodged against the mouth of the sewer.

Just how much sewage leaked into the sump while the station was running is not known. The one sample taken indicates that the amount was small. During shutdowns, however, and when the sewage had backed up in the sewer, it is evident that the sewage accumulated in the sump in concentrated amount, depending upon how low the water had been pumped in the sump and the head on the sewer. These periods of shutdown occurred each day the station was in operation. The scheme of operation at this station was as follows: Saturday, 11 a.m. to 6 p.m.; Sunday, 8 a.m. to 4 p.m.; Monday, 8 a.m. until noon, when for four hours the supply was pumped into the Pasadena distributing system at the lower end normally fed from Villa St. reservoir and another source; Tuesday, 8 a.m. to 1:30 p.m., when the station was shut down on account of this outbreak. When in operation, about 1,350 g.p.m. was pumped from the station.

It is curious to note that no illness came from pumping this water into the Pasadena system Monday afternoon. This may be accounted for by the fact that any polluted water accumulated during Sunday night had been pumped into the South Pasadena system earlier in the day. A sample at Villa St. reservoir taken Tuesday showed but 22 bacteria per c.c. on agar and no B. coli.

Seventeen cases of typhoid and one case of paratyphoid were reported in South Pasadena to May 26, and one case of typhoid from a part of Alhambra that uses the same water supply. The attack of intestinal disturbances disappeared as quickly as it came.

Arching Not Effective in Slightly Arched Gravity Dams

Relatively Small Curvature May be Harmful Because of Reduced Stability—Arch Action Cannot be Counted on Under Such Conditions—Construction Costs Increased by Arching

By B. F. Jakobsen
Consulting Engineer, San Francisco, Calif.

THERE HAS come into existence during the last twenty-five years or so a belief that it is always beneficial to curve a gravity dam. This belief amounts to almost an article of faith with some engineers, and they must find it extremely convenient when they become suddenly charged with the design of a great dam and realize their lack of ability to investigate the stress distribution in such a complicated structure. This simple and very plausible doctrine does away with all necessity to investigate stresses and reduces the design of a dam to the very simple proposition of making the base about two-thirds of the height and curving the dam in plan.

It is decidedly true that the stress determination is only one of the details necessary to a successful design, but it is a very important detail if an economic structure is to result.

When the curvature is considerable, there can be no doubt as to the benefit due to the curvature, but it does not follow that this holds good also when the curvature is relatively small. In the following it will be shown that the small amount of curvature sometimes given large dams is not beneficial, but harmful. For illustration I have used a large proposed dam, but a fairly typical example of this type of structure is the dam of the Tellassee Power Co. development on the Yadkin River at Baden, N. C. In describing this dam Taylor and Brehmer in "American Hydro-Electric Practice" state: "Although curved in plan to a radius of 1,678 ft., and supported by rock foundation on the sidehill, arch action was not considered in the dam design. Vertical contraction joints in a radial direction are introduced by constructing alternate blocks 45 to 50 ft. long, each block designed as a pure gravity section."

An inspection of Fig. 2, which has similar characteristics, shows that no part of the dam lies downstream from the downstream toe of the highest section (line *X-X*) and the great radius used, 1,678 ft., makes it fairly certain that the arch does not contribute materially to the stability of the dam. Whatever value the dam has it derives from the fact that it is a gravity dam.

A dam may fail from three general causes: (1) Lack of stability against overturning and sliding; (2) failure of the material of which the dam is constructed; (3) failure of the foundation proper.

The only manner in which failure of a properly designed gravity dam can occur, provided the material is sound and the foundation itself does not fail, is from excessive uplift. In that case the dam would revolve about the downstream toe of its highest section, admitting water under the dam, and this would continue until either the toe is fractured by crushing and shearing and the dam overturned, or the friction is so far reduced as to cause failure by sliding.

When a dam such as shown in Fig. 2 revolves about the downstream toe of its highest section, the whole upper part of the dam is lifted off its foundation—provided the contours are fairly parallel and the side slope fairly gentle—and the arch does not get a chance to act until sliding has actually taken place. If the dam does not slide, but fails by overturning, the arch may not come into action at all.

This cannot happen to a dam such as is shown in Fig. 1, where a fair portion of the dam lies farther downstream than the toe of the highest section (*Y-Y*).

An arched dam divides the water load between cantilever action and arch action in the ratio of the respective deflections. If the rigidity of the cantilever is for example 20 times that of the arch, the arch will carry only about 5 per cent of the water load, while the cantilever carries the 95 per cent. In this case the cantilever and the arch cannot act together, except to the very limited extent indicated. If failure does take place, the dam fails first as a gravity dam; the water enters under the foundation and if the arch prevents overturning, the friction has been so far reduced that the arch must sustain nearly the entire water load. If the arch is not capable of withstanding practically the entire water load without developing destructive stresses, the arch is of no real value.

Curving the axis of the dam affects the structure in the following manner: (1) It procures the advantage of arch action, its amount depending upon the particular design; (2) it brings the center of gravity downstream; (3) it brings the downstream toe downstream, except at the crown section; (4) it brings the center of uplift pressure downstream.

In order to compare a gravity dam curved slightly in plan with a straight dam, the dam shown in Fig. 2 will be investigated. It is 300 ft. high, the canyon width on top is 1,320 ft. and the radius of the upstream face is 1,500 ft.

Referring to Fig. 2, the length of the mean chord is at a depth h ft. below the top of the dam:

$$L = 2[m + (H - h)\tan\alpha] \qquad (1)$$

The lower case m and b refer to the general case, the upper case M and B to the specific. R is upstream radius; b is thickness of dam h feet from the crest.

The half central angle ϕ is:

$$\sin\phi = L/2R_{av} \qquad (2)$$

$$\tan\alpha = 510/300 = 1.7$$

The mean radius is:

$$R_{av} = R - b/2 \qquad (3)$$

The horizontal area of any section h ft. from the top is:

$$A = 2\phi b R_{av} \qquad (4)$$

and in this case $$b = 0.645h \qquad (5)$$

so that $$A = 2a\phi h(R - b/2) \qquad (6)$$

The center of gravity of this horizontal area lies a distance Z_a from the center C of the dam:

$$Z_a = (R_{av} + b^3/12R_{av})\sin\phi/\text{arc}\,\phi \qquad (7)$$

The most convenient manner in which to determine the center of gravity of the entire structure is by dividing it into several horizontal segments and deter-

mining the volume and center of gravity of each segment and the moment about C. The result is given in the table herewith.

The center of gravity lies at a distance (see table), $Z_g = 906.05/0.63389 = 1429.3$ ft. from the center C of the dam. $B = 0.645H$ and $H = 300$ ft. so that $B = 193.5$ ft. and since $R = 1500$ ft. the center of gravity lies $1429.3 - (1500 - 193.5) = 122.8$ ft. from point A in Fig. 2. The average radius at the base is then $1500 - 96.8$ or 1403.2 ft. and the central angle for point D is

$$\sin\phi' = 150/1403 = 0.107$$

and $\phi' = 6$ deg. 8 min. The point D in Fig. 2 lies

$$1306.5(1 - \cos\phi') = 7.45 \text{ ft.}$$

downstream from A.

As a check on the above computations, the entire volume of a straight dam is:

$V = 1020BH/6 + 300BH/2 = 9{,}873{,}000 + 8{,}711{,}000 = 18{,}584{,}000$ cu.ft.

The volume of the arched dam is 633,890 times 30 or 19,017,000 cu.ft. As indicated in the table, the area is also given in per cent and so is $\text{arc}\phi/\sin\phi - 1$; these two multiplied by each other give the excess in volume of the arched dam over the gravity dam. This gives 1.13 per cent, while above was found about 2.13 per cent or one per cent too much. This is as close as could be expected and quite close enough.

If the dam was straight, the center of gravity of the central part would lie $.667B$ from the downstream toe; the center of gravity of the other part lies $.75B$ from the toe of the highest section and the center of gravity of the entire dam lies therefore: $(9.873 \times 0.75 + 8.711 \times 0.667)B/18.584 = 137.6$ ft. from the downstream toe. Here $B = 193.5$ ft. and the volume of the straight dam is 1.13 per cent less than the volume of the arched dam.

As shown above the center of gravity of the arched dam lies 122.8 ft. from point A and 122.8 + 7.45 ft. from point D in Fig. 2 or an average of 126.5 ft. The use of the average in this connection gives an advantage to the arched dam that it may not be quite entitled to. The stability of the straight dam is proportional to 137.6 and the stability of the arched dam of equal weight is proportional to 126.5, so that the stability of the arched dam is only 92 per cent of that of the straight dam.

VOLUME, CENTER OF GRAVITY AND MOMENTS OF SEGMENTS

h	$H-h$	R_{av}	$\sin\phi$	$\text{arc}\phi$	A Sq.Ft.	Z_a	$M10^{-6}$	A Per Cent	$\left(\frac{\text{arc}\phi}{\sin\phi}-1\right)\times 100$
15	285	1495	0.424	0.438	12,690	1450	18.4	2.00	3.30
45	255	1486	0.393	0.404	34,800	1445	50.35	5.00	2.80
75	225	1476	0.361	0.369	52,600	1442	76.00	8.30	2.20
105	195	1466	0.329	0.335	66,700	1440	96.1	10.52	1.88
135	165	1457	0.296	0.3006	76,100	1433	109.2	12.01	1.55
165	135	1448	0.262	0.2651	87,000	1430	124.5	13.71	1.18
195	105	1437	0.228	0.230	83,300	1425	118.8	13.13	0.88
225	75	1428	0.195	0.196	81,400	1420	115.6	12.83	0.51
255	45	1418	0.160	0.160	74,600	1418	105.9	11.78	....
285	15	1408	0.125	0.125	64,700	1408	91.2	10.22	....
					633,890		906.05		

It may be justly argued, that the gravity dam is quite sufficient all by itself so long as no uplift exists and therefore it is not quite fair to neglect uplift when making this comparison, since arching the dam also reduces the moment due to uplift.

If uplift is assumed varying uniformly from a maximum at the heel to zero at the toe, since this is the usual and generally accepted assumption, it is easy to see that this uplift pressure may be represented as the weight of a structure exactly like the dam (triangular section), or as the dam itself but constructed of proportionately lighter material. It follows therefore that the center of uplift pressure must lie directly below the center of gravity of the dam, and this was determined above.

The forces acting on the dam produce three moments taken about the toe of the highest section, as follows: M_w due to the weight of the dam; M_p due to the water pressure on the upstream face of the dam; M_u due to uplift.

As a measure of the stability of the dam we may take $M_w - M_p - M_u$, since this is the moment which would be required above the already existing forces to overturn the dam. Expressed in percentage of M_p we have:

$$k = \frac{M_w - M_p - M_u}{M_p} \qquad (8)$$

In order to have definite ideas, let the uplift at the heel be assumed as 50 per cent of the water pressure

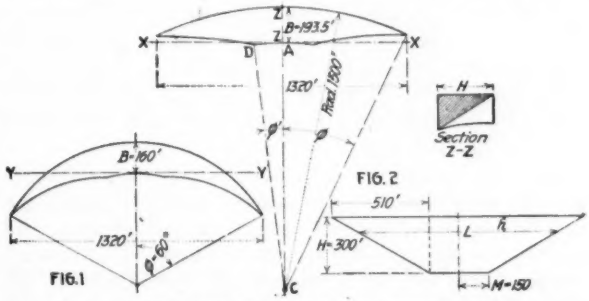


FIG. 1—DIAGRAM OF DAM WITH SHORT RADIUS
FIG. 2—DIAGRAM OF DAM WITH LONG RADIUS

and as zero at the toe. The uplift for a straight dam is then:

$$U = 2\left[62.5 \times 0.25 \times H \times 0.645 \times H \times 150 + \int_0^H \frac{62.5 \times 0.645}{4} h^2 1.7\,dh\right]$$

$$= 272{,}000{,}000 + 308{,}000{,}000 = 580{,}000{,}000 \text{ lb.}$$

The weight of the dam, assuming concrete to weigh 150 lb. per cubic foot is $18{,}584{,}000 \times 150 = 2{,}788{,}000{,}000$ lb., and the difference between the weight and the uplift multiplied by the lever-arm which is 137.6 ft. is $M_w - M_u$ and gives $304{,}000 \times 10^6$ ft.-lb.

The water pressure produces a moment:

$$5.2 \times 1020 \times 300^3 + 10.4 \times 300 \times 300^3 = 228{,}000\ 10^6 \text{ ft.-lb.}$$

so that

$$k = \frac{304 - 228}{228} = 0.333$$

For the arched dam, assuming the same weight and uplift as for the straight dam, M_w and M_u are both 92 per cent of their former value while M_p remains the same. Then

$$k = \frac{280 - 228}{228} = 0.228$$

This represents a reduction of about one-third of the excess moment available with the straight dam. Disregarding arch action this is a fairly good measure of the disadvantage of arching this particular dam.

There remains to be investigated the advantage derived from arch action, generally claimed for any arched dam. The amount of arch action will depend upon the relative deflections of the cantilever and the arch and these are therefore first investigated. As

has been shown by the writer in the *Transactions* of the American Society of Civil Engineers, Vol. LXXXIV (1921), p. 99, the deflection due to shear cannot generally be neglected. The total deflection of a triangular section is x ft. from the base, so that $x = H - k$.

$$y_c = \frac{62.5H^3}{EB^3}x^3 + \frac{90.2H}{EB}(Hx - x^2/2) \qquad (9)$$

Here $H/B = 1.55$ and E is in lb. per square foot. The first part of equation (9) is due to bending, the other part is due to shear. For $E = 144 \times 3{,}000{,}000 = 432{,}000{,}000$ the deflection at the various elevations are as follows:

h	x	y_c
290	10	0.0010 ft.
250	50	0.0058 ft.
200	100	0.0135 ft.
100	200	0.0346 ft.
0	300	0.0631 ft.

Let the arch stresses be investigated for $h = 100$ ft. here $b = 64.5$ and the average radius is 1,468 ft. and the half central angle is $\sin \phi = (150 + 1.7\ 200)/1468 = 0.334$ and $\phi = 19$ deg. The stress in this arch computed by the ordinary cylinder formula is

$$s = p \times r/b = 43.4 \times 1500/64.5 = 1010 \text{ lb. per square inch.}$$

Due to the fact that the central angle is small, the ordinary cylinder formula cannot be relied upon and it is necessary to compute the stresses by the exact formulas derived by Professor William Cain in his paper "The Circular Arch Under Normal Loads" in the *Transactions* of the American Society of Civil Engineers, Vol. LXXXV (1922), p. 233.

The ordinary cylinder formula was derived as an approximation for thin pipes, such as for example penstocks, and no one thinks of applying it to heavy cylinders, as for example to gun barrels, since it is known that it will give stresses only one-half or perhaps only one-fourth as great as the actual stresses. The extension of the cylinder formula to apply to arched dams could be defended only by the lack of a correct formula, such as Professor Cain has now derived, and moreover the approximation should only be applied where the thickness is relatively small compared to the radius and at the same time the central angle is fairly large, say 120 deg. or over.

Computing the stresses by the exact formulas we get: At the crown 1558 lb. per square inch compression and 211 lb. per square inch tension, at the abutments 2450 lb. per square inch compression and 1068 lb. per square inch tension. This is in place of the uniform compression of 1010 lb. per square inch computed from the cylinder formula. If there is any shrinkage due to the setting of the concrete or the lowering of the temperature, the compression at the abutments will be less and the tension greater. The tensions exist at the extrados of the abutment and at the intrados of the crown.

The concrete cannot withstand any such tension as this and will crack where the tension is considerable. As a consequence of this a secondary arch is formed which lies entirely within the original arch but is free from tension, as explained by L. J. Mensch in discussing Professor Cain's paper. This secondary arch may be found at least approximately by drawing arches of smaller radii within the given arch and applying Professor Cain's formulas to these arches until an arch is found that is not stressed in tension. For illustration two arches were investigated; for one the average radius is 1012 ft., $b = 24$ ft., $b/r = 0.0237$ and the central angle is 56 deg. This gives at the crown 2450 and 1122 lb. per square inch compression and at the abutments 3120 and 470 lb. per square inch compression. Since this gives a fairly high compression at the extrados of the abutments another arch was tried, as follows: The average radius is 1065.5 ft., $b = 31$ ft., $b/r = 0.0291$ and the central angle is 52 deg. 50 min. The stresses are then at the crown 2180 and 638 lb. per square inch and at the abutments 2965 lb. per square inch compression and 119 lb. per square inch tension. Therefore, on the supposition that no shrinkage takes place in the concrete, the maximum compression in the arch will be about 3000 lb. per square inch and not 1010 as found above.

The stresses found by the method of secondary arches are likely to be an under-estimate for the reason that the water load is actually upon the original arch.

It may be argued that the flowing of the concrete under stress and the fact that the modulus of elasticity decreases with the stress both act to equalize the stresses throughout each section and that for this reason the actual stress will be less than the 3000 lb. per square inch determined above. This may be quite true, but until we can show at least approximately to what extent a lowering of the stress does take place, it is not good practice to allow a substantially higher maximum safe stress than one would otherwise do. Also the fact pointed out above, that no allowance has been made for shrinkage stresses, should be considered.

It may also be contended that the arch will act more nearly as a hinged arch than as an arch fixed at the abutments and that in this way the stresses will be much smaller than those found for a fixed arch. For this reason the original arch $b = 64.5$ ft. was investigated. As before the average radius is 1468 ft. and $b/r = 0.044$; the stresses at the crown are 1570 and 266 lb. per square inch and 1000 lb. per square inch at the abutments.

The assumption that the arch is hinged is however an entirely unsafe assumption. An approximate computation shows that when the arch is fixed at the abutments, the moment at the abutments is sufficient to produce a turning of the rock surface of only about 20 per cent of the turning which takes place in the arch at the abutments when the arch is hinged. If it were actually hinged, there would be no turning moment produced at the abutments, which would turn the rock surface as required. It is therefore unsafe to assume that the arch is hinged and the assumption that the arch is fixed at the abutments is in this case substantially correct. At least no other interpretation is compatible with a safe and conservative design.

The deflection at the crown section of these arches when they carry the full water load is 0.875 ft. for the arch with an average radius of 1065.5 ft. and fixed at the abutments and 0.69 ft. for the hinged arch. The cantilever deflection at $h = 100$ ft. is only 0.0346 ft. giving a ratio of 25.3:1 for the fixed arch and 19.9:1 for the hinged arch. It is evident from this that the arch cannot in this case enter into action until after the cantilever has been broken by a very excessive deflection. The stresses throughout this dam, when computed as an arch dam lie between 2,000 lb. per square inch and probably 3500 lb. per square inch and it there-

fore follows that if this dam fails, it will fail first as a gravity dam; during this period the arch will be of practically no value. When the cantilever has been broken the whole water load would come on the arch and would, as shown, cause destruction stresses, and failure of the dam as an arch dam.

Conclusions—The arching of the axis of this dam considerably decreases the stability and the arch action does not take place until the cantilever is—or at least is likely to have been—completely broken and the arch stresses are so excessive that no reliance can be placed upon arch action. Arching in this case therefore means that a definite disadvantage, the reduction in stability, is exchanged for a very problematical gain due to arch action. If the dam is built in short sections and if by properly arranged and nearly frictionless contraction joints, each section may be considered entirely independent of every other section so that the dam will not act even partly as a whole, then it might be said that the arching is not outright harmful. In reality no dam is built in that fashion and therefore arching would be a disadvantage.

To this should be added that arching a dam increases the cost per cubic yard. It may not be easy to agree on the exact amount, but the writer has discussed this phase of the subject with a contractor who has built some very large dams and he was of the opinion that the additional cost of forms, of construction plant and of handling the material was by no means negligible.

A dam to fit this site should evidently be built either as a straight gravity dam, in which case it could have a somewhat larger volume for the same money, or it should be curved to a radius much smaller than 1500 ft., and this should be done especially if considerable uplift is to be feared at the site in question. And moreover the radius should be decreased towards the bottom so as to insure a fairly large central angle throughout the dam. Whether the section is reduced below a gravity section or not I believe is quite immaterial so long as the stresses are within reasonable and safe limits, when properly computed.

New Subterranean Railway Station at Rouen

Special Correspondence

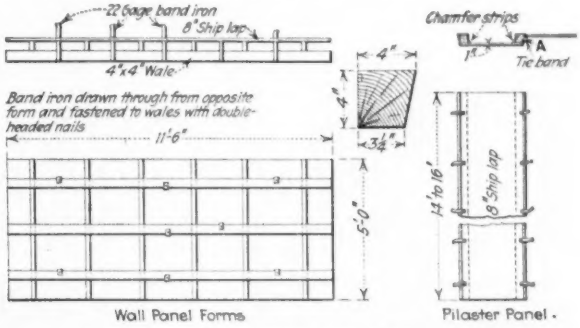
The rebuilding of the railway station at Rouen, France, now nearing completion, has been particularly difficult due to the fact that trains on the main line from Paris to Le Havre, numbering about 100 daily, could not be interfered with. The station proper is below the street level and the reconstruction necessitated the demolition of a tunnel about 300 ft. long, the building of retaining walls 720 ft. long containing 8,500 cu.yd. of masonry, and the moving of 39,000 cu.yd. of rocky soil. The principal structure of the new station is the 100-ft. viaduct which carries the Rue Maladerie over the tracks. It is made of reinforced concrete arched in form, but supported by pillars between the tracks. It is skewed in relation to the railroad track and required a complicated system of scaffolding to avoid interfering with the movement of trains through the station. The upper surface is covered with a light coating of slag, which in turn is covered with a thin coating of cement before the backfilling is put on. This arrangement provides for drainage.

Unusual Forms for Concrete Paper Plant

By Dan S. Howman

Superintendent of Construction,
C. R. Meyer and Sons Co., Oshkosh, Wis.

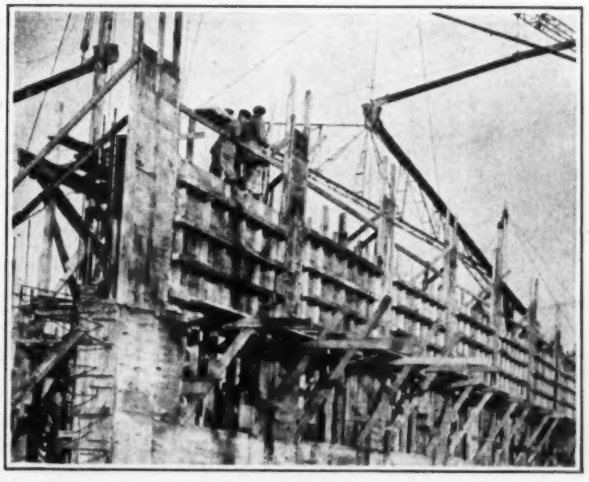
In building the reinforced-concrete plant at Appleton, Wis., for the Riverside Fibre and Paper Co., rather unusual wall and pilaster forms were used, wherein maximum reuse was had and excessive breakage avoided. Separate inner and outer pilaster forms



DETAILS OF PILASTER AND WALL FORMS

were built of panel type. They were from 14 to 16 ft. in height. Special care was taken in aligning, plumbing and bracing them.

Intermediate wall forms were then built up in panels to a 6-ft. height and set in place between pilasters. These forms were held by a ⅞x3-in. strip cleated to the mitered side of the pilaster forms. A 1-in. band or hoop iron was used instead of the usual wire tie.



FORMS IN PLACE

After pouring, wall panel forms were loosened and raised, the pilaster forms remaining in place. Panels were stripped by cutting the several bands and removing the strip, the mitered side of the pilaster form allowing the necessary clearance. A short 4x4-in. gin-pole was used from the scaffold above to raise the panels. All panels were numbered for position. By frequent oiling very little breakage resulted. Panels were used from five to seven times without repair.

The building was designed by Orbison & Orbison, engineers of Appleton; the general contractor was the C. R. Meyer and Sons Co., Oshkosh.

From Job and Office

Hints That Cut Cost and Time For the Contractor and the Engineer

Brush Mattress and Mud-Cell Revetment Protects Missouri Pacific Ry.

UNDER peculiar conditions making it necessary to prevent river bank erosion and to check the sliding movement of a railway embankment near the river, the Missouri Pacific Ry. has used a combined brush mattress and mud-cell revetment with successful results. This construction is shown in Figs. 1 and 2. In the winter of 1918 erosion of a strip about 1,000 ft. long just west of Dover, Mo., caused the bank to settle completely out from under the track, resulting in suspension of operation until the embankment could be restored

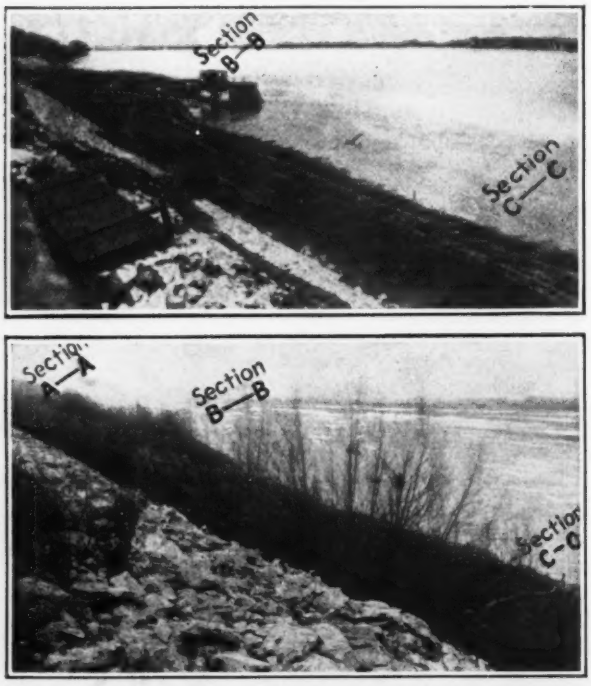


FIG. 1—BANK PROTECTION ON MISSOURI PACIFIC RY.
Above: cellular brush work on mattress, with riprap stone at left. Below: same location after cells had filled with river silt and willows had started growing in the brush.

with a cinder fill. At this point the line runs parallel with and close to the Missouri River.

A gradual and steady movement or flow of the material at the foot of the embankment was observed, with corresponding settlement in the track. Investigation of the strata forming the high bank of the river showed a layer of saturated sand near the water's edge, and a steady but slow movement into the river was observed for several days. The river was then approximately 16 ft. below the top of the river bank, the depth of water being 10 ft. The caving of the river bank and the sliding out of the embankment caused the stoppage of several drains through the latter, and allowed the runoff from the bluffs to seep through the embankment. This resulted in a thoroughly saturated bank from top to bottom and made any adequate method of surface drainage impossible. The difficulties of correcting these adverse conditions were increased by a heavy snowfall which covered the embankment for about a week.

It was apparent that to stabilize the embankment and provide drainage it would first be necessary to secure a foundation upon which to build. The river bed was composed of shifting sand with no rock near enough to the surface to be reached either by excavation or by scour. This precluded dumping stone at the foot of the slope or the building of a toe wall. Piles were not given serious consideration, for when driven into the sand and silt they would be subjected to scour by the river and to displacement and overturning by the lateral pressure of the embankment. Some form of subaqueous foundation upon which other work could be placed was necessary and with this in view a heavy "loose fascine" type of willow mattress was constructed for a distance of 1,400 ft. along the foot of the bank.

This type of mattress is constructed by placing poles 16 to 24 ft. long, parallel with the bank and spaced 8 ft. c. to c. The poles average 4 to 6 in. in diameter at the butt and 2 to 4 in. at the tip. Fascines or rolls of willow brush with an average thickness of 14 in. when bound, are then placed at right angles to the poles, pulled down hard with "come alongs" and bound over and under with longitudinal "sewing cables" of ¾-in. galvanized strand, spaced 8 ft. c. to c. Each cable is turned around the bottom pole and fastened with a 2½-in. staple. Lateral or anchor cables of ½-in. galvanized strand are run through the mattress at right angles to the sewing cables and have a turn around each of the bottom poles. These anchor cables, spaced 20 to 40 ft. c. to c., as required, are run up the bank through trenches and attached to deadmen placed 8 ft. or more back of top of bank and 4 ft. below the surface of the ground.

The mattress is begun at the upper end of the work and constructed continuously downstream. It floats during construction and when completed is sunk by ballasting with rip-rap stone. After about 100 ft. of mattress was completed, mud cells consisting of pole cribs or cells 6 ft. square, bound at each intersection with ⅜-in. galvanized strand, were started on top of the mattress, as shown by section A-A in Fig. 2. These cells are extended downstream for a distance of 240 ft. or to about the upper end of the slide. For 980 ft. from this point mud cells as shown in section B-B were constructed of poles and willow brush mixed. At each 3 ft. in height the cells were stepped back, as shown in section C-C, to conform approximately to a slope of 1 on 2. A fender or cover mattress was then constructed over the mud cell structure, extending down over the foundation mattress, and terminating at the lower end of the structure, as shown in section B-B. From this point to the end, 180 ft. below, a structure was built as shown in section C-C, but instead of a

fender mattress, brush decking was laid solid over the cells at each 3 ft. in height. Throughout the length of the high mud cells, wing walls of mud cells 16 ft. wide were led out of the structure and back to the high bank. These wings were spaced approximately 48 ft. apart, forming pockets behind the main structure for the reception of sediment.

After the mud cell and mattress work had been completed, the bank above the top of the mud cells near

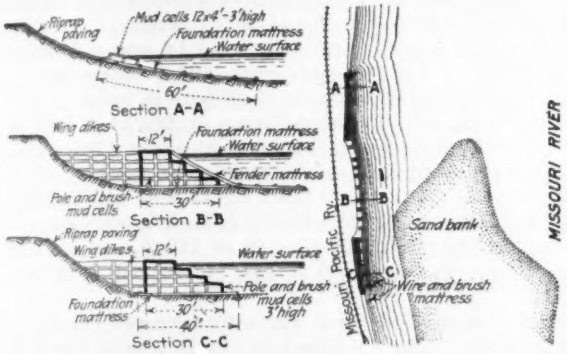


FIG. 2—MUD-CELL REVETMENT ALONG MISSOURI RIVER

the ends was graded and paved with riprap stone to prevent wave action against the top bank and possible cutting behind the structure. This paving was extended a short distance above and below the mud cells for the protection of the ends.

The condition presented in this case was that of an embankment slowly moving into the river which carried away the material as rapidly as it reached the water, and a river bed so shifting and unstable as to offer no satisfactory foundation. It was necessary, therefore, to place a foundation which could be sunk and made stable enough to support a superstructure which in turn would catch and hold silt and sediment, act as a retaining structure and also permit the drainage from the bluffs above and from the saturated embankment to reach the river at low water without displacement of the embankment or the structure. The structure described was designed with this idea in mind. The foundation mattress was built extra heavy to carry the load to be imposed, and the mud cells were carried to a stage of 20 ft. or 30 ft. above the river bed, the tops of the cells being built sufficiently high to allow for settlement which always occurs in such structures when they are filled and covered with silt. The wing walls which extended into and were a part of the main structure were carried back against the high bank, as anchors against overturning of the main structure, and to act as a drag against the main cells sliding out.

After the "June rise" in the summer of 1919 it was found that the whole structure was filled solid with silt and sand to a point half way to the top, and that the total settlement of the structure below the river bed as it existed at the time of construction was approximaely 6 ft. The settlement was gradual and even, and no breaks in either the wing walls or body of the work were observed. A heavy growth of small willow was springing from the willow poles and brush. Inspection after the next high water showed that the structure was completely covered with silt and that there was a sand bar heavily reinforced on the inside and with a dense willow growth on top. This sand bar has been acting as a banquette on the river side of the embankment, and being pervious to water, has afforded excellent drainage, especially during low-water periods, at which times the embankment has been found to be practically dry. No movement in either the old roadbed or in the berm formed by the protection work at the foot of the embankment has been observed since the structure first settled to a firm bearing on the river bed. Since the work has resisted the numerous rises in the river and the action of floating ice for approximately five years, it is now considered as successful. A recent inspection showed the work to be in perfect condition.

The mud-cell type of construction is not new, but probably it has never been employed before for the correction of such conditions as are described above. The work was designed and the construction supervised by W. G. Turner, assistant engineer in charge of river protection, under the direction of W. M. Neptune, principal assistant engineer, and E. A. Hadley, chief engineer, Missouri Pacific Ry. J. C. Bauer was inspector on the work for the railway. The Massman Construction Co., Kansas City, Mo., had the contract for this work. The cost of the 1,400 lin.ft. of work, including engineering, work-train charges and freight on material, was approximately $27,000, or a trifle less than $20 per foot. It is estimated that a fill of approximately 80,000 cu.yd. has been accumulated by sedimentation and held as a berm at the foot of the slope.

Repairing Broken Pipe in Creek Crossing

DURING an ice freshet in Cazenovia Creek a heavy stone previously frozen into the ice was dislodged, carried 100 ft. downstream and dropped directly over a pipe line of the Western New York Water Co., breaking it. The pipe, which is in 9 ft. of water, had become undermined due to excessive velocities occasioned by

COFFERDAM OF SAND BAGS AND SHEET PILES FOR REPAIR OF PIPE CROSSING

constriction of cross-section at a railroad bridge just downstream from the pipe crossing. The cofferdam shown in the picture was unwatered by steam-engine-driven centrifugal pumps, a traction engine located on the bank furnishing the steam. More recently, for such breaks the steam pumps have been replaced by gasoline-engine-driven pumps, materially reducing the cost of similar repairs.

Harry F. Huy, general manager of the company, states that this occurrence simply represents an ordinary incident in water-works operation, although many consumers think all the company has to do is to lay pipe and the water will always thereafter be delivered without expense.

Derricks for Steel Erection

IN THE ERECTION of tall steel buildings of the skyscraper type there is a decided preference for the guyed derrick, although stiff-leg derricks are used to a smaller extent, as indicated by an inquiry among contractors specializing in work of this kind. The size and shape of the building may be factors in the selection of type of derrick, but it appears evident that the selection is in part a matter of personal opinion and the desirability of using such equipment as may be available, since both types may be seen on structures of very similar dimensions.

Opinions in favor of the guyed derrick may be summarized as follows: (1) Less floor area occupied, with consequent greater facility for storing and handling material on the working floor; (2) faster by 20 per cent on a given piece of work; (3) easier to handle, to erect and to dismantle; (4) 25 per cent less time to raise or step up from floor to floor, using the mast as a gin-pole to raise the boom, and then using the boom as a gin-pole to raise the mast; (5) the boom is more easily swung; (6) a full circle swing of boom is available; (7) with mast and boom made in sections their length can be adjusted to the size of the building.

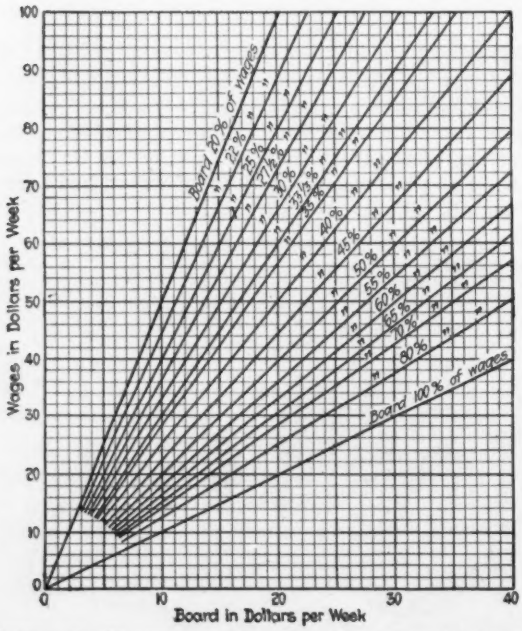
An ideal arrangement suggested by one contractor comprises a 110-ft. mast with 100-ft. boom and a minimum hoisting capacity of 10 tons in order to provide for unloading steel delivered on trucks as well as for handling the heaviest individual pieces. Some contractors use derricks of 15 to 20 tons capacity. One reply states that a longer boom can be used with a guyed derrick, thus covering a larger area of work. But the boom length rarely exceeds 100 or 110 ft. and stiff-leg derricks with booms 100 ft. and 115 ft. long have been used.

An objection advanced against the guyed derrick is that with its guys attached to the structural frame it subjects this frame to lateral stresses that were not contemplated or computed when the building was designed. In fact, some architects object to or even forbid the use of the guyed derrick for this reason. One erector states that the difficulty has been overcome by using wire cables for temporary lateral and diagonal bracing of the steel frame, which bracing has extended in some cases as much as six stories below the floor on which the derrick is seated. Furthermore, in open panel construction, where small tie beams are used and are insufficient to stand the thrust in the working floor, on which the derrick is seated, these beams may be reinforced by timbers (usually 8x8 in.) placed on top of the beams.

While one erecting engineer will not permit the use of guyed derricks for the reason that they will pull the building frame out of line, another engineer of equal experience states that this is simply a matter of proper and experienced handling. It is claimed also that the guyed derrick is not as safe or stable as the stiff-leg derrick, so that for erection done by the former it is specially important to have the work in charge of a careful foreman.

From experience with both types of derricks a contractor says that for a building of such size that two derricks may be used to advantage he prefers the stiff-leg derrick on account of safety for the men and the smaller amount of labor required to keep the steelwork plumb. With two stiff-leg derricks placed so that one can raise the other, a derrick can be shifted up to position on a new floor in four hours and with no more cost than the raising of a guyed derrick. On the other hand, one contractor estimates a saving of 25 per cent in time of raising a guyed derrick as compared with a stiff-leg derrick. Another says this saving is greater than the loss of time due to men working around the guys. Still a third contractor says that an experienced crew has stepped up a guyed derrick in half a day, while the similar shifting of a single stiff-leg derrick would take one and one-half to two days. On a building of relatively small area, the guyed derrick might be at a disadvantage owing to the steepness of the guy lines interfering with swing of the boom.

Relation Between Wages and Board



ONE OF *Engineering News-Record's* correspondents has suggested that construction superintendents might be interested in having printed a chart showing a relationship between wages paid and board charged now and ten years ago on construction projects so that workmen might see, graphically, what is a fair rate to charge for board. For instance, if a laborer received in 1914, $12 a week and paid $3 weekly for his board, he might expect to be charged $12 a week now on the basis of receiving a weekly wage of $48. Obviously other considerations govern board charged than the mere change in the value of the dollar. Labor might have been paid $12 and charged $3 for board in 1914 on an accessible job, but that labor should not expect the exact ratio to obtain today on an inaccessible one, where cost of transporting food to camp is excessive. However, the chart is offered for what clarity it may lend to such disputes in construction camps.

From Job and Office

Hints that Cut Cost and Time

From Job and Office

For Contractor and Engineer

It appears important that a guyed derrick for this kind of work should be of such construction that it is not necessary to unreeve the lines every time the derrick is stepped up to another floor. For this reason the load line sheave near the bottom must be on the outside of the mast. Furthermore, the derrick must be so arranged that the boom will stand vertically against the mast, so as to allow a full swing of the boom even if the guys are so steep as to limit its swing in a lower position. All guys should be provided with turnbuckles and shackles to permit of rapid attachment and adjustment.

Large stiff-leg derricks are commonly used in placing the exterior stonework of large office buildings and similar structures, even where guyed derricks are used for the erection of the steel frame. For this work the stiff-leg derricks may be placed on the roof, so as to cover the full height, or they may be placed lower down on one of the completed floors, with the mast on the outside and having its head held by a stirrup secured to the steel framing.

Hoist Engines Used as Stump Pullers

By A. D. Blakeslee

C. W. Blakeslee & Sons, Contractors, New Haven, Conn.

About Feb. 1 we were awarded a contract for clearing 90 acres of woodland for a golf course for Yale University. The wood was practically all second growth hardwood and ran up to 12 or 14 in. in diameter, with the average about 6 in. Our contract was to remove all the wood, stumps and roots to a depth of 8 in.

Sewer Outfall "Goes to Sea" Before Being Sunk in Place

The 460-ft. length of 44-in. riveted steel pipe which will constitute the outfall end of the temporary sewer now being built by the City of Los Angeles was recently launched, in one piece, towed to place and sunk in less than an hour. This length of pipe was riveted up on the beach, bulkheads were bolted on at each end to make the line watertight, and valves were put in for admitting water when the line was in position to be sunk. When the tide and wind conditions were right a steam tug towed the pipe out through comparatively quiet surf and with the aid of lines to a winch on a trestle over the site planned for the outfall, the pipe was maneuvered into place. Before sinking the pipe the shore end was pulled up the beach above tide level. This temporary outfall will carry the sewage that has been flowing down Ballona Creek. It is an emergency outlet to be used only about a year, by which time the city's new outfall sewer project is to be ready for service. A description of the Los Angeles sewer construction program appeared in *Engineering News-Record*, June 7, 1923, p. 1000. Construction of the temporary outfall as well as the main sewer is under the direction of H. A. Van Norman. Photographs show the pipe being rolled into the surf, towed to sea and then brought back to position beneath the trestle, ready to be lowered to place.

Our first thought was to equip the job with tractors, but, while waiting for the delivery of the first one, we tried out a double-drum steam hoisting engine and found that this worked to such good advantage that we have used four of these rigs, pulling down about one-half an acre of trees with each rig per day. We set the engine near a swamp, so as not to have to pump water, and then ran single lines out, running through snatch blocks where necessary and covering an area of about 1,400 ft. in diameter. We use a single horse to overhaul the slack and about four men to hook on the chains. We pull most of the trees with a single line, but for the larger ones double up and use one single block getting double purchase. Some of the trees pull over, but we are unable to pull them clear of the hole, so that we leave them and when the wood choppers trim them out they leave a 5-ft. stick on the stump. We later hitch on to these with a snatch block behind them and turn them out in the opposite direction. By this method, we are able to pull them all clear. We are using a 2-ton and a 3½-ton tractor to pull some of the smaller stumps and trees in isolated places, but are pulling the bulk of the work with the steam rigs.

We put some wood choppers in and cut the small growth up to about 4 in. in diameter, before we started pulling on these small stumps. We left about 18 in. of a stick so as to give us a chance to hitch on our chains. The larger trees we left and did not cut until we had pulled them down.

We have had no set height at which to make our hitches, finding that we had to vary the height greatly according to the kind of wood and the size of the trees.

When a hoister has pulled down everything within the reach of the 700-ft. cable, we put a snatch block about 25 ft. up into a tree, which we have left standing, and pull all of our stumps up to this tree and pile them around it.

When an area is cleared and the stumps piled, we simply run out our lines and hitch on to a tree, unfastening the anchorage from the rear of the hoister and go ahead, skidding the machine along the ground to its next setup.

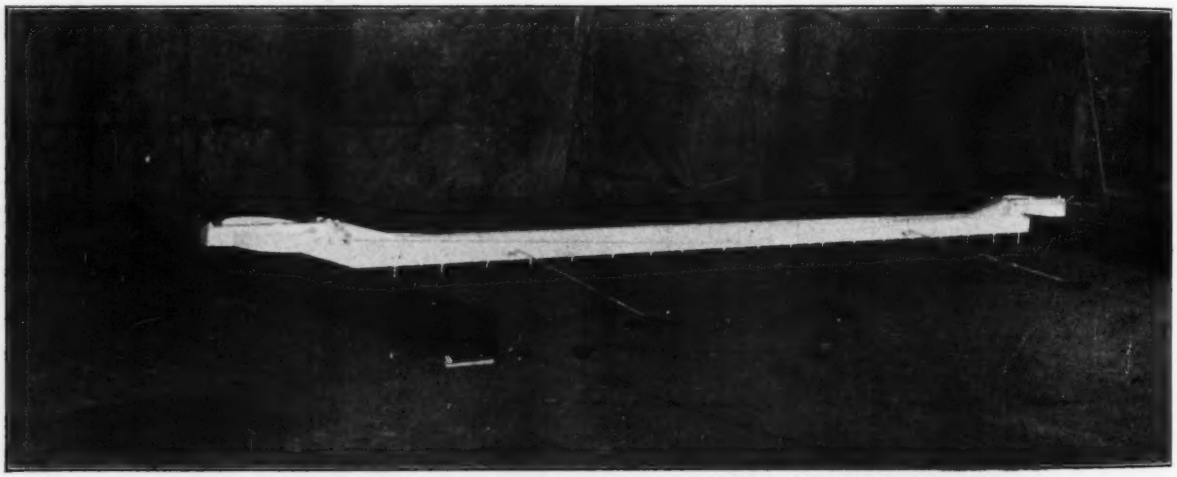
From Job and Office

Hints that Cut Cost and Time

Adjustable Subgrade Templet for 18-Ft. Concrete Pavement

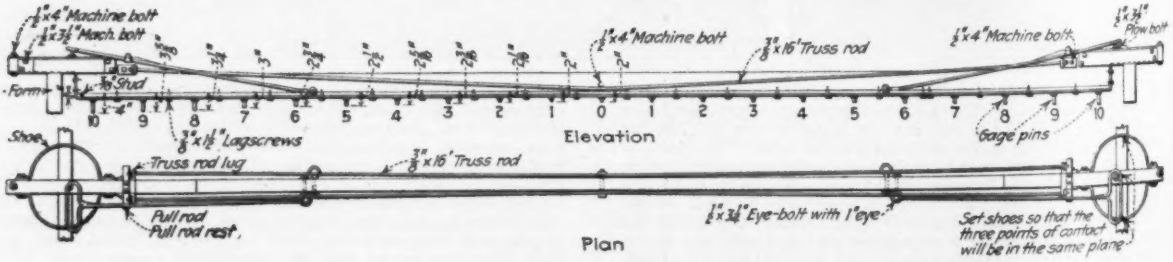
By F. M. Balsley
Engineer-Inspection, Wisconsin Highway Commission, Madison, Wisconsin

THERE IS a clause in the Wisconsin Highway Commission's concrete paving specifications to the effect that an approved templet, so designed as to describe the shape and depth of the subgrade, must be kept on the forms between the paver and the deposited concrete at all times. If properly used, this templet insures a slab of the specified thickness.



WORKING MODEL OF SUBGRADE TEMPLET USED IN WISCONSIN CONCRETE ROAD WORK

Considerable trouble has been experienced with templets that were of such light construction that the sag in the plank rendered them useless. No provision has been made in the old templets to keep the device perpendicular to the upper edge of the forms; conse-


½"x4" Machine bolt
½"x3½" Mach. bolt
Form
¾" Stud
10 9 8 7 6 5 4 3 2 1 0 1 2 3 4 5 6 7 8 9 10
Elevation
⅝"x1½" Lagscrews
Shoe
Truss rod lug
⅝"x16' Truss rod
Pull rod
Pull rod rest
½"x3½" Eye-bolt with 1" eye
Plan
½"x4" Machine bolt
⅝"x16' Truss rod
½"x4" Machine bolt
½"x3½" Plow bolt
Gage pins
Set shoes so that the three points of contact will be in the same plane

PLAN AND ELEVATION OF WISCONSIN HIGHWAY SUBGRADE TEMPLET

From Job and Office

For Contractor and Engineer

quently when in use the plank would cant to such an extent that the pins would not cut to grade.

For an 18-ft. road, the wood portion of the templet is cut out of a 3-in. x 8-in. x 20-ft. select fir plank, surfaced on both sides to a thickness of 2½ in. and notched out as shown in the drawing. The lower edge of the plank is shod its entire length with a ¼-in. x 2½-in. steel strap, drilled at intervals as shown for the gage pins and the lag screws that hold it in place. Two ⅝-in. adjustable truss rods, one on each side of the plank, tend to prevent sagging.

The gage pins are made of ½-in. bolts, threaded down and cut off to the length shown in the table on the plan. The lower end is squared to permit the use of an ordinary wrench in making adjustments. The gage pins are screwed into square nuts which are inset into the edge of the plank and they are held rigidly in place.

A ⅝-in. check stud is placed at each end of the templet over which a chalk line may be drawn as a guide in checking sag or making adjustments in the pin lengths due to wear or other causes. Loosening the jam nut underneath the strap makes it possible to raise or lower the pins or to remove them entirely.

This templet can be used to describe any shape or crown in the subgrade by inserting gage pins of the required length. The templet as shown in the plan is set for a slab depth of 7 in. at the center, 8 in. at the edge, laid on a subgrade having a 2-in. parabolic crown.

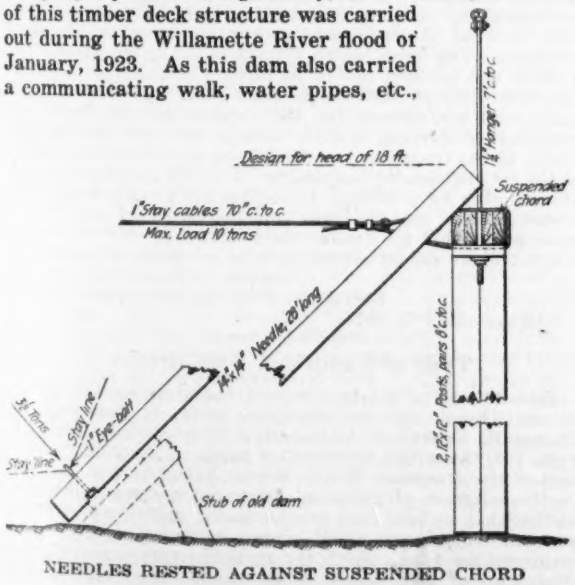
This templet weighs 175 to 180 lb. complete, and should not cost over $75.

Break in Timber Dam Closed During Flood

By J. C. Stevens

Stevens & Koon, Consulting Engineers, Portland, Ore.

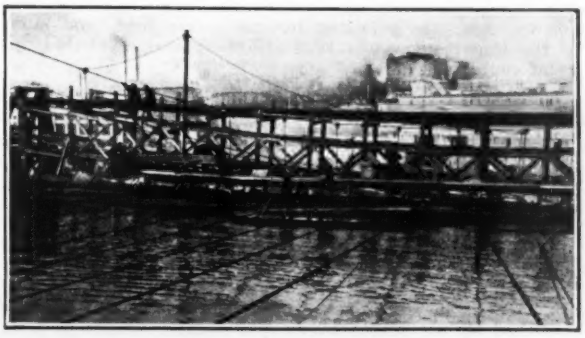
A NOVEL METHOD was used in closing a break in the forebay dam of the Hawley Pulp & Paper Company's plant at Oregon City, Ore. Ninety-five feet of this timber deck structure was carried out during the Willamette River flood of January, 1923. As this dam also carried a communicating walk, water pipes, etc., between the buildings it was necessary to make repairs quickly.


1½" Hanger 7'-2" c. to c.
Design for head of 18 ft.
Suspended Chord
1" Stay cables 70" c. to c.
Max. Load 10 tons
14"x14" Needle, 28' long
2-10"x12" Posts, piers 6' c. to c.
31 Tons
Stay line
1" Eye-bolt
Stay line
Stub of old dam

NEEDLES RESTED AGAINST SUSPENDED CHORD

The first work was the construction of a suspension bridge over the gap to establish communications. Plans were then prepared to close the break in the dam without closing the plant. This necessitated making the closure while the water was still high, otherwise there would have been no water for the turbines. The break was 95 ft. in length and during the period of closing, a head of from 14 to 16 ft. of water was pouring through it.

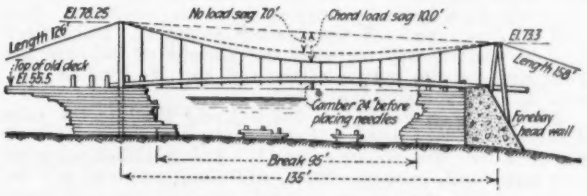
A heavy timber chord was suspended over the gap, and stayed upstream by means of cables anchored to



CLOSURE OF BREAK PRACTICALLY COMPLETED

the opposite wall. Fourteen by fourteen-inch needles, 28 ft. long, were then placed progressively from one end of the break, on a 1 to 1 slope with one end on the rock-bed and the other on the chord. After 14 needles had been set a vertical strut was placed under the chord in the quieter water behind them. By proceeding step by step the closure was effected without shutting down the plant and the cost was far below that at first contemplated.

The erection sag of the suspension cables, the loaded sag and the camber in the chord were all computed in advance so that upon completion the new deck of needles was in line with the broken ends of the forebay dam. The computation of stresses and loaded sag for the suspension cables was somewhat involved since the



DETAILS FOLLOWED IN CLOSING DAM BREAK

chord was not stiff enough to distribute the load uniformly. It was assumed that the chord would distribute the live-load to the hangers in proportion to their distance from the point of application. After the needles were in place and the leakage stopped a rock-filled timber crib was built behind them and the cables and suspension bridge removed.

The plan was devised and construction supervised by Stevens & Koon, consulting engineers. The work was executed by A. Guthrie & Co., contractors, of Portland, Oregon.

Letters to the Editor

This department aims to be a forum for the discussion of the views of engineers and contractors. The range of interest should be as wide as possible. Contributors are, therefore, asked to make their letters short.

Courtesy to the Visitor from Afar

Sir—Will you allow me through the medium of your valuable paper to express my appreciation and gratitude to the American engineers of the cities I have visited during the past three months for the great kindness and for the facilities provided for me to see first-hand many of the important works that are being carried out in this great country.

I am commissioned by the city of Auckland, New Zealand, to visit the States and England with the purpose of seeing the methods adopted to carry out municipal works.

I have proved that I came among friends, and in every instance I found the responsible men very busy, but they had time to supply me with such information and data, and they made the necessary arrangements for me to inspect many important works that are being carried out.

I leave the States with a very great admiration for the engineers who are entrusted with making provision for the needs of your large cities. Their courage, ability and achievement in spite of the many difficulties will long be cherished by me, and I count it a great privilege to have met so many during my stay.

JAMES TYLER,
Assistant City Engineer, Auckland, New Zealand.

New York City, May 23, 1924.

Pressure on Sewer Pipe in Trenches

Sir—After reading the editorial comment on p. 515 of the March 27 issue of *Engineering News-Record*, one hesitates before he dares bring up any objections to the article by George C. D. Lenth, on the "Relation of Sewer Trench Width to Load on Pipe." Yet, the writer can hardly agree with the wide conclusions of the editorial that the problem is solved by the tables resulting from Mr. Lenth's computations and desires to bring his objections to the attention of those designers who are going to use the tables without realizing the limitations of them.

The theory upon which these tables are based may be summarized in the statement that the vertical pressure acting upon a pipe set in a vertical trench of the minimum possible width is equal to the weight of the loose backfill less the frictional resistance along the sides of the trench resulting from the unequal settlement of the undisturbed and backfilled earth. Just after backfilling this will undoubtedly be a correct picture of the actual conditions. But will the load upon the pipe remain constant? The writer believes that it will increase, the amount of increase depending upon the nature of the soil and the kind of surcharge loading above the pipe.

The curves on p. 533 are qualitatively similar; the only difference between sand and clay is the numerical value of the pressure. The writer doubts it. The backfilled material will settle and, in most places, additional material will be added at the surface to prevent the formation of ruts, especially if the trench is in a roadway. The backfill tends to become part of the natural soil; age and water affect various materials in different ways in this tendency. A material like dry sand will, in a short while, lose all independent identity and recombine with the undisturbed portions. The writer is assuming the non-existence of "arching action," which phenomenon may be eliminated by the impact of traffic or other vibrations. Backfill materials with more cohesion than dry sand will maintain their identity for a longer period of time and prevent the full load from acting upon the pipe. Inspection of the curves, Fig. 11, for example, shows that the pressure on an embedded pipe in some materials is given as less than the weight of the material directly above the pipe.

In one of Dean Marston's series of tests, a 40-in. culvert embedded under almost ideal conditions under 20 ft. of a sandy loam top-soil (96 lb. per cubic foot) showed a load of slightly over 6,200 lb. just after backfilling was completed. The load increased almost uniformly with age to 7,300 lb. per foot in three months. The weight of the material directly above the culvert was 6,240 lb. The same pipe covered by a similar fill by ordinary methods of covering a culvert showed an initial load of 10,800 lb., increasing (not uniformly) to 12,257 lb. in a little less than 3 months. Corresponding results were obtained for smaller heights of cover. If the load to be sustained by a pipe embedded in a sandy loam top-soil is not less than the weight of the material directly above it, such values should certainly not be used for dry sand or any other soil of low cohesive strength. The danger of a loose backfill need not be stressed—any one who has had to repair streets and roads on "settled" trench fills knows the economy of right methods.

In enumerating the methods of pipe laying and relative bearing values (Figs. 1 to 8), the author has omitted the case of pipe set into a bed of loose or pulverized earth, thereby obtaining a uniform bearing and permitting a further settlement of the pipe, after the backfill has reached equilibrium with the sides of the trench. In this connection, the writer wishes to bring attention to some recent experimental work by Emil Bie, reported before the Brooklyn Engineers' Club on Nov. 8, 1923. Although on a rather small scale, it was shown that with sufficient settlement a pipe embedded in sand or in garden soil sustains no load, even though surcharges are placed on the surface of the fill.

Experiments by George L. Fowler on the strength of culvert pipe resulted in the conclusions that the maximum pressure on a pipe is reached under a definite depth of cover, and at greater depths the pressures due to surface loading decrease. It was estimated that a depth of cover equal to three times the diameter of the pipe would protect it from any increase of pressure due to surface loading. The tests were on a full sized scale.

In conclusion, the writer desires to emphasize the fact that in dealing with soils, or rather with any material which is neither solid nor fluid, general laws cannot (at the present state of our knowledge) be formulated, and no design can be based upon any one set of tests or tables. The actual conditions should be carefully studied to determine the worst possible case, and a somewhat less favorable case should then be assumed. Where facts are definitely known, full advantage should be taken of them; but, if different reliable sources disagree in results, can final conclusions be drawn before the differences are explained?

JACOB FELD,
Professional Engineer, New York City.

Brooklyn, N. Y., April 7, 1924.

[This letter was submitted to Mr. Lenth, who replied as follows.—EDITOR.]

Sir—I think it is high time that the engineers of this country accept the findings of investigations of scientists and the conclusions which are arrived at only after thorough consideration. I think that this rather mildly states what was done at Ames, Iowa, under the direction of Dean Marston. The point which I tried to bring out and which I think was brought out in the article in question was that the rule-of-thumb methods so many years in vogue by engineers and contractors for the construction of sewers in trenches of varying widths without any relation to the width of the trench or the size of the pipe should be superseded by engineering principles. Few of the engineers of this country have utilized Dean Marston's work and as his investigations are the only investigations of any great moment on this subject, at least until his conclusions are disproved, we should accept them in our engineering work.

GEORGE C. D. LENTH,
Secretary, Clay Products Association.

Chicago, May 7, 1924.

Type of Pump and Pipe Breaks

Sir—Regarding the opinions of engineers on the merits of centrifugal and reciprocating pump as requested by George H. Fenkell in *Engineering News-Record*, April 24, page 737, the writer presented a paper at the 1923 convention of the American Water Works Association at Detroit, on the selection of pumping equipment and made the prediction that in less than ten years the centrifugal type of pumping equipment would almost entirely supersede the reciprocating type. With the marked improvement in the efficiencies of both steam turbines and centrifugal pumps

in recent years, this type of equipment now compares very favorably with the reciprocating type of equipment.

It is a known fact that the reciprocating pump does cause pulsations in water lines and these pulsations are naturally more detrimental to a system than if the water were delivered at an even flow and pressure. With the centrifugal type of pump it is possible instantly to cut off a large area of the discharge main without damaging the pump, as it simply churns the water in the casing while a reciprocating pump will of course,, be damaged seriously by such procedure. Surges and pulsations in the line will be entirely eliminated by the use of the centrifugal pump, which will decrease maintenance and repair expense in the water mains.

ARTHUR M. MULLERGREN,
Kansas City, Mo., May 6, 1924. Consulting Engineer.

Why Washington Is Not Flooded

Sir—Your issue of May 22, 1924, p. 911, under the title "Floods in the Potomac" gives an explanation as to why Washington is no longer flooded when the river is at high stage, that is so far from accurate as to prompt the following brief outline of the improved drainage and flood prevention works at Washington insofar as this special feature is concerned. As these works have been in operation for more than fifteen years, successfully functioning without interruption during that period, perhaps it is not strange that their detail should be forgotton even by the technical press. They were designed and constructed under the writer's immediate supervision as engineer in charge, in accord with the recommendations of the Board of Sanitary Engineers, consisting of Rudolph Hering, Samuel M. Gray and Frederick P. Stearns, appointed by President Harrison immediately after the 1889 flood.

Briefly, the plan provided for segregating the "low area" of the city from upland drainage, the construction of dikes to keep the river out, and a deep storm sewer for the low level drainage delivering the flow to storm pumps in the main sewerage pumping station located on the water front at the foot of New Jersey Ave., S. E. The portion of the city flooded in 1889 is included in the "low area," nearly one and one-half square miles, extending on either side of Pennsylvania Ave. from the foot of Capitol Hill to the Treasury at Fifteenth St., a distance of more than a mile. This area has been protected from river in-flow by elevating the streets across two low valleys, on the south and west. near the water front, forming, in effect, low protecting dikes. These, by the way, have not yet been raised to the elevation called for in the plans. Regardless of river stage, all rainfall within this area is delivered by the B St. and New Jersey Ave. trunk sewer to the storm pumps housed in the main pumping station.

This portion of the equipment includes eight 100-sec.-ft. storm pumps reinforced by three 100-sec.-ft. combined sewage and storm pumps. The plant is designed for a maximum runoff of 900 sec.-ft. and maximum lift of 15 ft., with hydraulic datum 3 to 10 ft. below normal river stage, M.H.T. depending on the volume of storm flow. Independent of river stage a hydraulic grade on the suction side of the pumps, of not less than three feet below mean tide, is maintained.

This, perhaps, will explain why Washington was not flooded during the recent Potomac freshet.

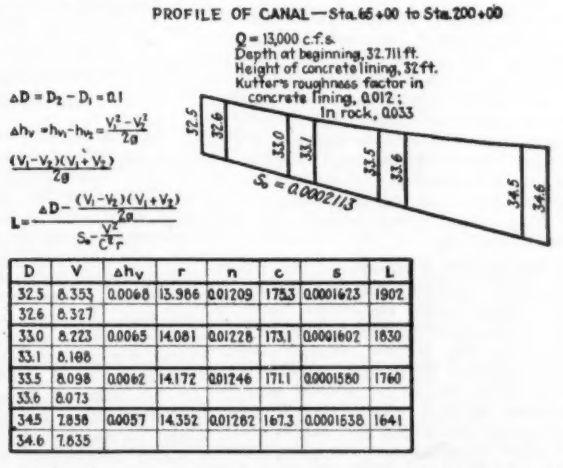
Your article concludes with the statement, "Contributing factors which have increased the immunity of Washington from floods by accelerating the runoff are the confining of tidal creeks at the foot of Capitol Hill, and James Creek a mile further uptown, to the sewer system." This statement in incorrect. The "tidal creeks at the foot of Capitol Hill" (there was only one, known as "Tiber Creek") were "confined to the sewer system" twenty years before the 1889 flood. As to St. James Creek, a puny stream below the Capitol and not uptown, it was canalized nearly 50 years ago to afford a new outlet for the Tiber sewer, and, later, under the flood prevention plans, this canal was filled and abandoned to P St., S., near the water front where the south dike was built. The 30-ft. span Tiber sewer, having been relieved of most of its drainage, was then connected to the B St. and New Jersey Ave. trunk sewer, to afford a storm-water reservoir on sudden, peak discharges, so as to steady the hydraulic grade line and give additional time for starting the storm pumps. Of course all this had nothing whatever to do with "accelerating the runoff," but the contrary, and, if it had, would not have been of the slightest avail in "increasing the immunity of Washington from floods."

These floods have been caused by the river rising to a stage above street elevations in the heart of the city, most frequently with no accompanying local runoff, and never materially affected by local rainfall. To those unfamiliar with Washington topography, it might be remarked that Pennsylvania Ave., although one to two miles back from the river bank, between the Capitol and Treasury, is practically dead level and 3 to 6 ft. below the recorded river level at Key Bridge during the recent rather moderate freshet stage. A study of the river data indicates that a maximum stage of 20 ft., at Key Bridge can be anticipated, or about 6 ft. higher than the May, 1924, record.

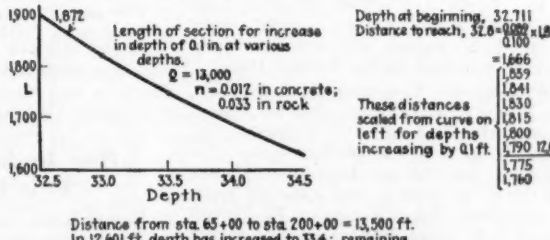
ASA E. PHILLIPS,
Consulting Engineer.
Washington, D. C., May 30, 1924.

Computing Backwater and Drop-Down Curves

Sir—Methods of computing backwater and drop-down curves were described by Alva G. Husted in *Engineering News-Record*, April 24, p. 719. The writer has done a considerable amount of work of this kind during the past six


PROFILE OF CANAL—Sta. 65+00 to Sta. 200+00
Q = 13,000 c.f.s.
Depth at beginning, 32.711 ft.
Height of concrete lining, 32 ft.
Kutter's roughness factor in concrete lining, 0.012; in rock, 0.033
32.5
32.6
33.0
33.1
33.5
33.6
34.5
34.6
S_f = 0.0002113
$\Delta D = D_2 - D_1 = 0.1$
$\Delta h_v = h_{v_1} - h_{v_2} = \frac{V_1^2 - V_2^2}{2g}$
$\frac{(V_1 - V_2)(V_1 + V_2)}{2g}$
$L = \frac{\Delta D - \frac{(V_1 - V_2)(V_1 + V_2)}{2g}}{S_e - \frac{V^2}{C^2 r}}$

D	V	Δh_v	r	n	c	S_f	L
32.5	8.353	0.0068	13.986	0.01209	175.3	0.0001673	1902
32.6	8.327						
33.0	8.223	0.0065	14.081	0.01228	173.1	0.0001602	1830
33.1	8.198						
33.5	8.098	0.0062	14.172	0.01246	171.1	0.0001580	1760
33.6	8.073						
34.5	7.858	0.0057	14.352	0.01282	167.3	0.0001538	1641
34.6	7.835						



Distance from sta. 65+00 to sta. 200+00 = 13,500 ft.
In 12,601 ft. depth has increased to 33.4; remaining distance to sta. 200 = 899 ft.
Increase in depth = $\frac{899}{1,775}$ x 0.1 = 0.051; depth at sta. 200 = 33.451

DIAGRAMS FOR COMPUTING BACKWATER AND DROP-DOWN CURVES

or eight years, both on natural streams and on regular artificial channels. He has used methods the details of which were suggested by R. D. Johnson of New York. A diagram based on Mr. Johnson's suggestion was presented in a paper by T. H. Hogg, hydraulic engineer of the Hydro-Electric Power Commission of Ontario, read before the Engineers' Club of Philadelphia some two years ago, and published in *Engineers and Engineering*. The method has this advantage over the one suggested by Mr. Husted, namely, that the changes in depth used in successive sections of the channel for which the length is computed are so small as to overcome any objection that might be urged against the assumption that the Kutter or Manning formula is applicable.

J. J. TRAILL,
Engineer of Tests, Hydro-Electric,
Toronto, Ont., May 9, 1924. Power Commission.

News of the Week

CURRENT EVENTS IN THE CIVIL ENGINEERING AND CONTRACTING FIELDS

News Brevities

Prior to Adjournment, both Houses of Congress passed the bill increasing to three years the limitation in connection with the filing of claims by shippers against the transportation companies. The measure simply equalizes the statute of limitations. Under previous law, the carriers could file claims for undercharge against shippers for a period of three years, while the statute of limitations as applied to shippers' claims against the carriers became operative at the end of two years.

State Engineer Dwight B. La Du, of New York, started June 5 for an inspection trip of the Barge Canal in the middle and western part of the state. He will first inspect the canal in the western division from Rochester to Buffalo, then the middle division which radiates from Syracuse. While in Buffalo he will direct the preparation of plans for dredging a portion of the old Erie Canal lying within the city limits of Buffalo.

A House Bill Authorizing the remission of all unpaid customs duties on War Department materials returned from France after the armistice was passed by the Senate shortly before adjournment. This law eliminates a bookkeeping charge of several hundred thousand dollars which the Treasury Department made against the War Department on account of the phraseology of the Tariff Act which provided that exported commodities which were changed in nature abroad must pay duty if returned to the United States.

Without the Necessity for a roll call, the House of Representatives on June 5 passed the Federal aid highway bill, authorizing a $75,000,000 program for the fiscal year to end June 30, 1926 and for the fiscal year to end June 30, 1927. In addition, the bill carries $7,500,000 for each of those fiscal years for forest roads and trails. No effort was made to secure the passage of the bill in the Senate. However, by passing it early in the December session no delay will result as the actual appropriation must be carried in the next session's agricultural appropriation bill.

Delaware River Bridge Contract 16, providing for construction of sewers in Philadelphia required for the Delaware River bridge, was awarded by the bridge commission on June 6, 1924 to Lombardi Co., Inc., of Philadephia, upon a low bid of $173,464.25. The sewers are to be located along the line of the bridge from the bulkhead line at the Delaware River west to Fourth Street. They comprise sections of reinforced concrete, brick, and vitrified pipe sewer construction. Bids were opened on June 4.

Ohio River Bridge at Evansville Is Planned

Federal aid for the construction of a bridge across the Ohio River at Evansville is asked by the advocates of a bridge at that point. At present there is no highway connection over the Ohio between Louisville and Cairo, a distance of several hundred miles. The proposed bridge would be a link in the Dixie Bee Line Highway, connecting Chicago with Nashville by way of Terre Haute and Evansville. The available highway moneys of both states are sufficient for covering any or all of the cost of the bridge. Of the estimated cost of $3,000,000, it is hoped to have the federal government pay half and the two states each one-fourth.

Western Society Bestows Award on A. N. Talbot

At the annual meeting of the Western Society of Engineers June 9, Prof. A. N. Talbot received the Washington Award, founded by John W. Alvord and bestowed on "an engineer of some reasonable degree of professional skill or administrative attainment who has contributed most largely to the public welfare." The award is in recognition of Prof. Talbot's life work as student and teacher, investigator and writer, and for his enduring contribution to the science of engineering.

Dr. Elwood Mead, Commissioner of Reclamation, the principal speaker of the evening, spoke on "The Relation of Land Settlement to Irrigation."

The newly elected officers of the society are as follows: President, E. T. Howson, western editor, *Railway Age;* vice-presidents, Homer E. Niesz, F. E. Morrow and Major Rufus W. Putnam; treasurer, George W. Hand.

How Reclamation Relief Is to Be Applied

Regulations covering the deferment of payments on reclamation charges, under the relief act of May 9, have just been issued by Elwood Mead, Commissioner of Reclamation. The regulations emphasize the fact that the relief act is not intended to authorize deferment of charges by those who are able to pay them, but is to apply only to those cases which really need relief. Pending passage of the general reclamation act revision now before Congress, the relief act will be applied only for short-time extensions (limit March 1, 1927). Written requests for relief are to be submitted by applicants to the project chief clerk; applications rejected or only conditionally approved by the chief clerk are to be referred to the director of finance at Denver, with final appeal to the commissioner and to the Secretary of the Interior.

Cement Dumping Case Is Heard in Washington

Pacific Coast and Southeastern Makers Charges Denied—Conditions Investigated Further

Washington Correspondence

Charges that foreign producers of Portland cement are dumping their product in the United States at prices below the home market value were entered and denied at a hearing before Assistant Secretary of the Treasury McKenzie Moss June 6. All parties at interest were given to July 15 to file briefs. A decision is expected from the Treasury Department late in August or early in September. The hearing was attended by sixty producers, importers and attorneys.

H. A. Hayward, in charge of the dumping law section of the customs division, told the conference that although the conference has been investigating charges of cement dumping for more than a year, its investigation has not been completed and that the evidence is not conclusive in all cases. Summarizing the results of the investigation to date, he stated that as to Norway sales here have been made at less than the home market value; that the manufacturers of Sweden and of Germany both refuse to permit examination of their books; that as to the United Kingdom, it is extremely doubtful that there has been dumping; that as to Denmark, comparisons with home market value have been extremely difficult because of the differences in the size of the lots sold; that as to Belgium, in general the prices of cement sold for export to the United States have been higher than the home market value, but that there should be further investigation in this country.

Invoices Produced

All Pacific Coast manufacturers and those of the southeastern states except those in Oregon were represented at the hearing by Jesse P. Crawford, a Washington attorney. It is the domestic producers of these two sections along the seaboard who are suffering because of the alleged dumping, Mr. Crawford stated. He produced a number of invoices in support of his charge of dumping and declared that investigation has shown that shipments from Norway have been sold for export to the United States at 75 cents a barrel below the home market price; Sweden, $1.15 below the home market price; Denmark, 37 cents, and Belgium, 40 cents. Imports in the first four months of 1924 from these four countries have totaled 615,470 barrels, he said.

In behalf of the importers a general denial of dumping was entered, it being alleged that sales of imported cement are made at small profit and sometimes even at a loss.

Important Reclamation Appropriations Lost

House Conferees Fail To Agree Upon Senate Amendments Written Into Second Deficiency Bill

Refusal of the House conferees on the Second Deficiency Appropriation bill to agree to a Senate amendment to appropriate $800,000 for the Spanish Springs irrigation project in Nevada led to a filibuster by Senator Pittman, of Nevada, in the last thirty minutes of the first session of the 68th Congress, on June 7, which resulted in failure to enact the entire bill and also resulted in blocking enactment of other last-minute legislation, including the special appropriation for administration of the bonus bill.

The conferees labored all afternoon on the last day of the session in an effort to reach an agreement on the numerous amendments which the Senate had written into the deficiency bill. The House conferees particularly objected to numerous Senate amendments having to do with reclamation and irrigation projects. The Senate receded from a number of its amendments and the House also withdrew some of its objections, repeated reports being made to each body by the conferees, but as the hour set for adjournment approached, the two Houses still were at odds on the Spanish Springs project, from which the Senate conferees were ready to recede and on the Senate amendment authorizing a salary of $10,000 a year for the director of the Bureau of Reclamation and fixing his qualifications.

Last Minute Effort Fails

In a last-minute effort to save the bill, Senator Warren, chairman of the Senate conferees, made an attempt to recede on these two amendments, but Senator Pittman insisted upon the bill being recommitted with instructions to retain the Spanish Springs project, especially in view of the fact that there had been no disagreement on other projects which admittedly were less worthy and which had not gone through investigation and approval.

Senator Ashurst, of Arizona, declared that the action of the Senate conferees in receding from the amendment appropriating $200,000 for operation and maintenance and completion of construction of the irrigation system in part 1 of the Mesa division of the Yuma project in Arizona and various other sums for that project was a much more serious matter than the situation regarding the Spanish Springs project because it was a continuation, and not a commencement, yet rather than have the entire bill fail, he joined in asking Senator Pittman to withdraw his objections, but without result.

Previously, the conferees had by gradual steps reached an agreement reducing from $1,500,000 to $375,000 a Senate amendment for the Salt Lake Basin project in Utah; reducing from $1,250,000 to $315,000 a Senate amendment for the Owyhee project in Oregon; reducing from $1,500,000 to $375,000 a Senate amendment for the Yakima project in Washington; approving Senate amendments appropriating $800,000 for the North Platte project in Nebraska and Wyoming; and establishing a new set of regulations for irrigation projects; and abandoning Senate amendments appropriating $250,000 for the Warm Springs project, Oregon, $200,000 for the Yuma project, $597,000 to reimburse Yuma owners for maintenance of the Colorado front work and levee system, $100,000 for the continuation of this system, and $50,000 for a survey of the Carlsbad project.

Municipal Bond Sales in 1924 May Be $1,000,000,000

Based on total sales of $535,000,000 of state and municipal bonds for the five months ending May 31, and an expected falling off during the summer, *The Bond Buyer* estimates that the total sale of these bonds for 1924 will be about a billion dollars. The highest sales for the first five months of a year during the past ten years was $564,000,000 in 1922 against a minimum of $113,000,000 in 1918.

First Cables Erected on Delaware River Bridge

The first cables between towers of the Delaware River Bridge connecting Philadelphia and Camden were erected during the past week. These cables consist of twelve 2¼-in. wire ropes, and will be used to support the erection footbridges for use of workmen during the building up and compacting of the main cables of the bridge. The first four of the ropes were carried across the river on June 5, the operation being accomplished by towing across the river a barge upon which four reels of the wire had been set up. The wires were dropped to the bed of the river to avoid interference with navigation during the placing of the other strands across the river and the completion of other preliminary work.

The hoisting to the tops of the towers was accomplished on June 10. The ropes used for these footbridges will later be cut into lengths to be used for suspender wires in the structure. Wire for the main cables is now being spliced into long lengths and wound on reels in readiness for the cable construction operations.

Senators To Visit Irrigation Project in Nebraska

Authority for a subcommittee of the Senate Committee on Irrigation and Reclamation to visit the tri-county irrigation project in Nebraska during adjournment of Congress in order to report on the practicability of this project was contained in a resolution offered by Senator Norris and adopted by the Senate June 7. The project comprises the counties of Gosper, Phelps and Kearney and extends into Adams County, Nebraska, and would use water from the Platte River. The Reclamation Bureau has already made a survey and investigation.

State Sanitary Engineers Meet at Cincinnati

Various Phases of Water Supply and Sewage Disposal Form Main Topic of Discussion

Eighteen states were represented by as many engineers and the United States Public Health Service had four representatives present at the conference of State Sanitary Engineers held in Cincinnati, May 26-28. The main topics for discussion were the various phases of water supply and sewage disposal, the control of the milk supply, mosquitos, public bathing places and tourist camps.

The officers elected for the ensuing year were: Chairman, W. H. Ferguson, Illinois; vice-chairman, M. Z. Bair, Arkansas; secretary, George Buttman, Missouri; corresponding secretary, A. P. Miller, U. S. Public Health Service, Washington; and executive committee, W. L. Stevenson, Pennsylvania, and E H. Ehlers, Texas.

The Committee on Ground Water Supplies presented a detailed report with many recommendations designed to safeguard the quality of water from wells and with numerous references to intestinal outbreaks, chiefly typhoid, that had been traced to various defects in the protection of well supplies.

Tourist Camp Inspection

In a discussion of tourist camp sanitation, a summary was given of the inspection of 116 tourist camps or parks so scattered throughout Indiana as to make 2,200 miles of travel necessary for their inspection. The survey showed that of these camps, the water supply was good in about 73 per cent and bad in 27 per cent; that for the same 116 camps sewage disposal was good in only 22 per cent of the camps and bad in 78 per cent; while as to garbage disposal it was found that conditions were good in 47 per cent and bad in 53 per cent of the camps.

A survey of the milk supply of Missouri municipalities during 1923, stated George W. Putnam, state sanitary engineer, showed that in the five largest cities of the state milk control is in charge of full-time city employees so that the objective hereafter will be to secure improvement in milk supplies of places from 20,000 down to 7,500 population. The field work is being carried out under the direction of Mr. Putnam, with the aid of an assistant engineer and the co-operation of a veterinarian of the U. S. Public Health Service.

To Develop Susquehanna Power

Much more power is available in the Susquehanna River above Harrisburg than had been estimated, a report made by Stone & Webster reveals. The examination was made for Frank M. Waring of Tyron, Pa. The Federal Power Commission has authorized the issuance of a license to him covering this project. It has been asaumed that only 20,000 hp. could be developed, but the detailed study made by Stone & Webster indicates that 50,000 hp. is available.

Congress Committee Reports on Reorganization

Public Works Centered in Interior Department But Army Engineers Are Left as Before

The Joint Committee on Reorganization of the Senate and the House of Representatives, which was created under joint resolution on Dec. 17, 1920, and amended May 5, 1921, submitted on June 3 its report on the reorganization of the executive departments of the government. Inasmuch as Congress adjourned on June 7, no action was taken on the report.

The committee had before it the recommendations on government reorganization made to it in February, 1923, by President Harding, which recommendations were the result of the investigation by a presidential committee. The report of the joint committee parallels in some degree the Harding reorganization plan, the major difference being that the joint committee does not recommend the consolidation of the War Department and the Navy Department in one Department of Defense and does not change the status of the civilian activities of the Corps of Engineers of the United States Army. In the Harding plan the river and harbor work, now the principal civilian duty of the Corps of Engineers, was taken from that body and transferred to the Department of the Interior.

New Department Recommended

The joint committee's recommendation includes also a new cabinet position known as the Department of Education and Relief, which would take over all the welfare activities of the government such as the Public Health Service, the various educational and medical institutions, the Pension Office, the Bureau of Education, soldiers relief, etc.

The joint committee's plan also follows the Harding plan to the degree that the Department of the Interior is constituted in effect a department of public works though retaining its old name. The function of this department is construed to be the conservation of the public lands and the resources which underlie them and the control of the public works. It would, therefore, have transferred to it from the Department of Agriculture the Bureau of Public Roads, from the Department of the Treasury the Supervising Architect's Office, and from the Department of War, the Board of Road Commissioners for Alaska (which would be abolished and its functions transferred to some other branch of the department). At the same time, the department would be divested of its non-public work and public domain activities, such as the Pension Bureau, the Education Bureau, etc., which would go to the new Department of Education and Relief, and the Bureau of Mines and the Patent Office, which would go to the Department of Commerce.

Under the plan, too, the Department of Commerce gets the two bureaus noted above, the Lake Survey Office and the Inland Coast Waterways Office from the Department of War and the National Advisory Committee for Aeronautics. The Bureau of the Census would be changed to the Bureau of Federal Statistics and would take over other statistical work now done by the government, such as the mineral production and water-borne commerce statistics. A bureau of transportation also would be created under the Department of Commerce.

Further, there would be set up an independent bureau of purchase and supply which would control all purchases for the government, an office of public buildings and parks in the District of Columbia, which would take the office of the Superintendent of State, War, and Navy Department Building and the office of Public Buildings and Grounds of the War Department. A separate Bureau of the Budget would also be established, reporting only to the President.

The following paragraphs relating to rivers and harbors work are interesting:

"The committee gave careful consideration to a recommendation urged by many that river and harbor work should be withdrawn from the War Department, where it is now handled by the Chief of Engineers, and placed in the Department of the Interior. It is true, of course, that at the present time the improvement of rivers and harbors and the construction and maintenance of river and port works is chiefly done in the interests of navigation and commerce and presents few if any military aspects; and this constitutes the basis for the view that river and harbor work should be removed from the jurisdiction of the Military Establishment.

"With this view, however, the committee is not in accord. The assignment of Army engineers to river and harbor work is at the present time the principal means whereby these officers can acquire the engineering experience necessary to fit them to meet the demands put upon them in time of war; and, on the other hand, there is a measure of economy in using the personnel of the Corps of Engineers on necessary public works of a non-military character. Although these matters are subject to violent differences of opinion, there is little doubt in the minds of the committee that the present arrangement tends to increase the value and efficiency of Army engineers, without material disadvantage, if any at all, to the government. River and harbor work seems to present special phases of engineering, which would make it difficult to consolidate or amalgamate the organization maintained for its prosecution with other engineering agencies of the government; and the committee finds itself unable to recommend any change at this time in the arrangement under which the major responsibility for the engineering maintenance of inland navigable waters and port works is lodged in the War Department."

Elmer T. Howson Installed As President of Western Society

On June 7, Elmer T. Howson, western editor, *Railway Age*, was installed as president of the Western Society of Engineers, succeeding Charles A. Morse. He is one of the youngest engineers to attain this honor and the first editor. Mr. Howson began his engineering career, after graduation from the University of Wisconsin in 1906, as assistant engineer with the Chicago, Burlington & Quincy R.R. From 1909 to 1911 he was division engineer for the La Crosse Division, leaving the railway in the latter year to go into editorial work as engineering editor with the *Railway Age*. He is now editor also of two other railway publications of the Simmons Boardman Co., the *Railway Maintenance Engineer* and the *Maintenance-of-Way Cyclopedia*. He has long been interested in society activities not only from the reportorial standpoint but for the advancement of the profession. He was chosen secretary of the original Development Committee of the American Society of Civil Engineers and has been a member of the Western Society Board of Directors since 1919.

Engineering Employment Shows Slight Recession

From a wide survey made by E. B. Miller, employment secretary of the American Association of Engineers, covering all sections of the country, engineering employment conditions are not such as to warrant excessive pessimism. Although summer is generally a good season for engineering employment, since the first of May there has been an unwarranted slump. However, since the first of June engineering employment conditions are starting on the upgrade.

The greatest demand at the present time exists for highway and municipal engineers with some construction in the public utility field. A healthy demand exists in the large centers for architects and structural engineers. This is particularly noticeable in the Eastern section. Mechanical, chemical and electrical engineers are least in demand.

Over 60,000 engineering graduates and undergraduates will be seeking employment or some sort of connections during the month of June. Opportunities for these men are not very abundant as commencements are approaching. Graduates in civil engineering are most in demand as the schools close. The demands come mostly from highways and municipalities and individual large construction projects. This field, however, does not offer permanency in employment. Quite likely at least half of these men that find positions this summer will be changing their employment before the end of the year.

Electrical engineers are probably second in demand at this time. Graduates in mechanical engineering will feel the business slump the most. Manufacturers and machinery organizations both large and small, who employ mechanical engineers are feeling the present temporary depression most, says Mr. Miller.

Highway Commission of Pan-America Starts on Tour

Engineering Delegates Spend Two Days in Washington Prior to Examining Roads Throughout the East

Thirty-eight distinguished engineers, representing twenty of the nations of South and Central America, started their tour of the road systems of the Eastern United States with a very crowded two days in Washington, June 2 and 3. The commission, made up of delegates of the South and Central American republics, together with some American engineers who are accompanying it as guides, is inspecting the roads in this country under the auspices of the Highway Education Board. The functions in Washington consisted in a reception by Secretary of State Hughes, a reception by the President, a tree planting and luncheon at the Pan-American Union, a visit to Mount Vernon, where a wreath was laid at the tomb of George Washington, a theater party, inspection of the experimental road and tests of the Bureau of Public Roads at the Arlington farm across the river from Washington, a luncheon at the Chevy Chase Club, a trip through the Bureau of Standards, and a dinner at the National Press Club. The party left Washington the night of June 3 en route for Raleigh, N. C., where the actual inspection of the roads of the country will begin. They attended that week the field demonstration of highway construction and maintenance, or the so-called outdoor road show, at Greensboro, N. C., under the direction of the American Road Builders' Association, and from there started through the South toward Minnesota, Illinois, and Ohio. The entire trip is to take about thirty days.

The names of the delegates and their positions in their home countries are as follows:

ARGENTINA

Manuel R. Balina: Director of roads and pavements, Buenos Aires.

Luis Eugenio Brenta: Division chief, Department of Bridges and Roads of the province of Buenos Aires.

Roberto Kurtz: Chief, Section Highways, National Department of Roads.

Juan Agustin Valle: Civil engineer in charge of northern zone of Argentine national highway system.

BOLIVIA

Santiago Aramayo: Collector of Internal Revenue.

Juan Rivero: Engineer in charge of building important motor road in Yungas region.

BRAZIL

A. F. de Lima Campos: Railroad civil engineer in the state of Rio Grande de Sur; chief engineer of the federal inspection work of the reclamation service.

Joaquim T. de Oliveira Penteado: Inspector-general of streets and roads of the state of Sao Paulo.

Theodoro A. Ramos: Civil engineer; professor of differential and integral calculus, Polytechnic School of S. Paulo (1918); participated in the construction of the Pedroith Avenue at S. Paulo; hydraulic installations in the works commemorative of the independence of Brazil at S. Paulo made under his direction.

CHILE

Fermin Leon Martinez: Chief engineer, province of Valparaiso.

Hector Vijil Olate: Editor of "Roads and Touring," the official magazine of the Automobile Clubs of Valparaiso and Santiago.

Santiago Marin Vicuna: Engineer; technical expert to the fifth Pan-American conference at Santiago.

COLOMBIA

Pablo Enrique Caicedo: Director-general of national roads.

Julio Fajardo: Formerly chief engineer of Department of El Valle; now in highway contracting work.

Alberto Roncallo: Member of board of aldermen of city of Barranquilla.

COSTA RICA

Ricardo Pacheco: Consulting engineer, light and power company of F. J. Alvarado & Co.; staff engineer of the firm H. T. Purdy.

CUBA

Adolfo Arellano: Head of the firm of Arellano & Mendoza, contractors.

Armando Valdes: Contractor; active in road construction since 1907; has built many important works, among which were the Central Highway in Pinar del Rio Province, the highway from Camaguay to Santiago de Cuba, etc.

DOMINICAN REPUBLIC

M. A. Cocco: Civil engineer.

ECUADOR

A. C. Hidalgo: Engineer.

Richard Frederic Antoine Muller: Hydraulic and electrical engineer; in November, 1909, entrusted by the government of Ecuador, S. A., with the task of organizing the school of engineering of the University of Ecuador; taught engineering subjects there for six years and served as consulting engineer for the government of Ecuador.

GUATEMALA

General Juan Bautista Padilla: Director General of Roads.

HONDURAS

Manuel Angel Zelaya: Civil engineer; built difficult sections interoceanic highway, later serving as sub-minister of War, sub-minister of Public Works, head of the Railway Commission, etc.

MEXICO

Frederico Garcia Cuellar: Resigned from the army as captain of engineers to aid in development of road building in Mexico City and throughout the republic; is now head of the National Construction Co.

Jorge Nunez: Head of a division in the Department of Communications and Public Works of the Mexican National Government.

NICARAGUA

Adolfo Cardenas: Civil engineer; Minister of Finance and member of High Commission; financial expert; now in charge of road work.

PANAMA

Leopoldo Arosemena: Engineer.

Thomas Guardia: Engineer.

PARAGUAY

Alejandro Bibolini: Civil engineer; in charge of public highways National Department of Engineers.

PERU

Carlos Oyague y Calderon: Author Peruvian road construction legislation; specialist in road construction in mountain section.

Federico Fuchs: Director of Highways, Peruvian National Government.

PAN-AMERICAN HIGHWAY COMMISSION WITH PRESIDENT COOLIDGE ON THE WHITE HOUSE GROUNDS, JUNE 2

SALVADOR

Manuel Lopez Harrison: Engineer; has had charge of maintenance and construction of various public roads and bridges in the republic of San Salvador.

URUGUAY

Donato Gaminara: Consulting Engineer; Dean of the engineering faculty, University of Montevideo; professor of ports and lighthouses in same institution; president of the Society of Engineers of Montevideo.

Agustin Maggi: Member of the faculty of engineers of University of Montevideo; has occupied post as engineer in the office of the highway department of the Minister of Public Works of Uruguay; now engaged as chief of department of engineers.

VENEZUELA

Dr. J. M. Ibarra Cerezo: Director of highways and aqueducts in Ministry of Public Works.

Dr. Alfredo Jahn: Civil engineer; director of the Greater Venezuelan Ry., the major railroad system of the country.

The party was conducted by S. T. Henry, assisted by W. C. John of the Bureau of Education. Aside from their position as members of the commission, the delegates have been named by the governing body of the Pan-American Union as a committee to formulate for their consideration a program for the official Pan-American Motor Highway Conference which was called at the international conference held at San Diego a year ago. It is anticipated that the conference will be held in one of the South American countries some time next year.

Portland Bridge Scandal Leads To Twelve Indictments

The grand jury investigating the conditions under which three bridge contracts were recently let in Portland, Ore., completed its work on May 31. Twelve indictments have been returned: two against Charles Rudeen, and one each against Dow Walker and John H. Rankin, all former county commissioners, for malfeasance in office. Mr. Rudeen, Mr. Walker and Robert Kremers were also indicted one count each for bribery.

Contractors who were awarded the contracts for the construction of the three bridges were also indicted on one count each for conspiracy in submitting collusive bids. These men are J. H. Tillman, awarded contract for construction of the Burnside bridge; C. J. Parker, president, and T. H. Banfield, treasurer of Parker and Banfield, awarded the Ross Island contract; and George A. Sears, president and secretary and C. G. Huber, vice-president and treasurer, of the Union Bridge Co., awarded the Sellwood bridge contract.

The trial of Dow Walker, one of the deposed county commissioners, on a charge of bribery in connection with the award of engineering to Hedrick and Kremers began on June 4. Previous reports on the Portland bridge situation were printed in *Engineering News-Record*, April 24, p. 740 and May 15, p. 873.

Public Hearing On Philadelphia Water Supply Held

At a public hearing in Philadelphia on June 6, support was given to the plan recently recommended by the Philadelphia Water Commission for securing supply from the Perkiomen-Tohickon project augmented by water from the Delaware River at Torresdale (see *Engineering News-Record*, June 5, pp. 961 and 968).

The recommended plan had been criticized by Fred C. Dunlap, former chief of the Bureau of Water, who held that water in the Delaware River at Torresdale is more polluted than Schuylkill River water because it receives sewage of Philadelphia and Camden, washed upstream by the tide. Mr. Dunlap recommended use of the upper Delaware River in conjunction with the Neshaminy Creek, and estimated that the cost of this project would be but $60,000,000.

At the public hearing, engineers of the Water Commission estimated that the Delaware-Neshaminy project would be more expensive than the Perkiomen-Tohickon, because pumping would be necessary to secure the required pressure. It was shown by city records that the bacterial content of the Delaware River is less than that of the Schuylkill, although the B. Coli content of the Delaware was greater in 1923 than for the Schuylkill River. Water Commission engineers stated that completion of sewage disposal plants along the Delaware River will greatly improve the conditions, and that the present pollution is not excessive.

It was also pointed out at the hearing that six to eight years will be required for construction of the new water supply system. Present sources of supply must be continued for that period, but when the Perkiomen-Tohickon supply is secured, the water from the Delaware River will be handled as a reserve source.

Carleton E. Davis, who resigned as chief of the Bureau of Water in 1923 after having held this position for ten years, fully endorsed the Water Commission's recommendations, and stated that the use of two sources of supply afforded greater security to the city.

Adverse Report on Hudson Bay Railway Revealed

Speaking before the Canadian House of Commons, D. W. McLachlan, formerly engineer in charge of dredging operations at Port Nelson, eastern terminal of the Hudson Bay Ry., drew attention to the fact that in 1917 he had made a report opposing any further expenditures on the Hudson Bay route. This report was made to the Department of Railways, but for some reason was never made public. Mr. McLachlan said that it would cost considerably more to ship grain by the Hudson Bay route than by the Great Lakes and Montreal; that liners could never successfully run into Port Nelson; that navigation of the strait and entrance into Port Nelson harbor would not be feasible for more than two months of the year; and that in the year 1917 Port Nelson was ice blocked until Aug. 29.

Random Lines

One Hundred and Five Varieties

Over a year ago this column started to list the titles of various aspiring individuals who, possessing none of the educational or experiential attributes of the engineer, have sought to magnify themselves by appropriating the term "Engineer" modified by some adjective which they felt described best their own peculiar qualities. So many interested correspondents have been of late sending in duplicates of titles already published that we give below, the full list already printed. Those marked with an asterisk have not appeared before.

Acoustical
Adhesive
Advertising
*Aerial
Amusement
Anatomical
Aquatic
Automobile
Automotive Painting
Belting
Body
Box
Brewery
Business
Cafeteria
Carbonating
Cash Register
Catalog
Caterpillar
Cleaning
Clothes
Clothing
*Condemnation
Correspondence
Cost Cutting
Development
Dehydration
Display
Domestic
Dress
Dust
Dwelling
Economic
Electric Sign
Experimental
Export
Exterminating
Feed Plant
Financial
*Floor
Food
Forgery Prevention
French Hat
Furniture
Garage
Home
Horticultural
Hospitalization
Hot Dog
Hotel
Household
Human
Hymn Book
Imaginative
Income
Insurance
Kitchen
Life Saving
Liquidating
Magnetic House-cleaning
Man
Mechanical Air
Membership
Merchant
Milk
Mosquito
Motion Picture
Mowing
*News
Odor
Office Equipment
Organization
Osteopathic
Packing
Pageant
Paint
Photographic
Police Signaling
Political
Practical Heating
Pressing
Printing
Projection
Psychic
Publicity
Revolutionary
Sales
Second Story
Shoe Surface
Short Story
Social
Socio-religious
*Soft-Drink
*Space
Standardization
Surface Protection
Trade Catalog
Travel
Trunk
Turf
Vault
Vocational
Window Shade
Wrapping
Wrecking

* * *

Getting Together

They have just thrown the temporary cables across the Hudson at the new Bear Mountain suspension bridge, and have hung a light footwalk on them. This walk, according to a local newspaper, is to permit the engineers from opposite sides of the river to come to the middle and talk over necessary details of construction.

* * *

Not Only By Architects

H. Van Buren Magonigle, the well-known architect, in a scholarly paper before the American Institute of Architects gently attacks his profession for a too great devotion to precedent which, he says, is not far removed from plagiarism. In one office, he remembers, this adaptation went so far that "after a particularly flagrant bit of cribbing" the boys used to say "the old man has been anticipated again."

Engineering Societies

Calendar

Annual Meetings

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York City; Annual Convention, June 18-20, Pasadena, Calif.

AMERICAN SOCIETY FOR TESTING MATERIALS, Philadelphia, Pa.; Annual Meeting, Atlantic City, N. J., June 23-28, 1924.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION, University of Pittsburgh; Annual Meeting, Boulder, Colo., June 25-28.

NEW ENGLAND WATER WORKS ASSOCIATION, Boston, Mass.; Annual Meeting, Rochester, N. Y.; Sept. 30 to Oct. 3.

AMERICAN SOCIETY FOR MUNICIPAL IMPROVEMENTS, St. Petersburgh, Fla.; Annual Convention, Boston, Mass., Sept. 29-Oct. 3, 1924.

The American Society of Agricultural Engineers will hold its annual meeting at Lincoln, Neb., June 18-20. The program is built around two general themes: research as fundamental to agricultural development, and relation of agricultural engineering to economic production. There will be sessions of the farm power and machinery, the reclamation and the farm structures sections, and a discussion of rural electrification.

The Central Illinois Section of the American Society of Civil Engineers was addressed June 4 at Champaign-Urbana, by G. F. Burch, bridge engineer, Illinois State Highway Department, on "Highway Bridges." T. L. Condron, consulting engineer, Chicago, in discussion, described by means of slides the design and progress of the Twenty-third St. viaduct over the Illinois Central tracks in Chicago.

Personal Notes

W. H. DURBIN, vice-president and assistant manager of the Terre Haute Water Works Co., has been made superintendent of the property, sale of which to the American Water Works & Electric Co. has been approved by the Public Service Commission of Indiana.

JOHN G. SULLIVAN, consulting engineer, and for several years with the Canadian Pacific Ry. as manager of construction and then chief engineer, is president of Sullivan, Kipp & Chace, Ltd., consulting engineers and constructors, a newly-organized firm which has established offices in the Nanton Building at Winnipeg, Canada. The vice-president is THEODORE KIPP, whose experience has been largely in connection with industrial plants. W. G. CHACE, treasurer, has been hydraulic engineer for the Hydro-Electric Power Commission of Ontario and chief engineer of the Greater Winnipeg Water District, with its 96-mile conduit from the Lake of the Woods. CHARLES A. MONKMAN, secretary, is a former construction engineer of the Canadian Pacific Ry. and has had experience in structural steel design and general construction work. HUGH MCNAIR, gas engineer with the new firm, was for ten years gas engineer for the Public Utilities Commission of Manitoba.

H. R. TREVOR, San Francisco, Calif., is now connected with Fred H. Tibbetts, hydraulic engineer on reclamation work, in the design of irrigation structures. Mr. Trevor was formerly associated with the Pacific Fruit Express Co. in structural work on reinforced-concrete buildings.

LEWIS N. WHITCRAFT has been appointed by the Portland Cement Association as district engineer in charge of its Philadelphia office, effective June 1. Mr. Whitcraft has been for eight years field engineer of the association attached to its New York office; prior to that he was highway engineer for Somerset County, Maryland, and chief engineer of the hydrated lime bureau of the National Lime Manufacturers' Association.

ANDREW O. CUNNINGHAM, for the past eighteen years chief engineer of the Wabash Railway Co., has severed his connection with that road and opened an office at 2037 Railway Exchange Building, St. Louis, Mo., for consulting engineering practice, specializing in river improvement work, involving revetments and dikes, structural steel work including bridges, foundations and grade crossing eliminations. Mr. Cunningham is a graduate in civil engineering of the University of Minnesota.

NICHOLAS ALTIERI, formerly engineer in charge of federal-aid road work in Lowndes County, Alabama, and recently municipal engineer at Fort Myers, Fla., is now connected with the Berthe Engineering Co. with headquarters in Charleston, Mo.

H. E. PALMER has resigned as highway engineer of Red Lake County, Minnesota, after four years in that position, and is now resident engineer at Winthrop, Minn., in charge of two construction projects for the Minnesota Highway Department.

L. S. DIXON, formerly with the Eddy Paper Corporation of Three Rivers and White Pigeon, Mich., as manager of engineering, is now connected with the Riordan Pulp Corporation, of Canada, as consulting engineer.

ALFRED NUEFFER, formerly assistant engineer with the Truscon Steel Co., has joined the organization of the McClintic Marshall Co., as contracting engineer.

W. J. WELLS has been appointed city engineer of Huntingburg, Ind.; he was formerly connected with the Roberts & Schaefer Co. of Chicago.

LEWIS P. BAILEY, Norristown, Pa., has severed his connection with the Baldwin Locomotive Works, Philadelphia, for which he was assistant engineer on construction work, to become field engineer for the Portland Cement Association.

Obituary

MAJOR-GENERAL SIR GEORGE SCOTT-MONCRIEFF, of London, England, died suddenly June 4 at the Zbassayn section on the Polish-German frontier, at the age of 69 years. He was on his way to Poland in connection with the work of the London Society for Promoting Christianity Among Jews. Sir George's engineering services included command of the Royal Engineers in the China Expeditionary Force of 1900, the post of chief engineer at Aldershot from 1909 to 1911, and director of fortifications and works at the British War Office during the World War.

CHARLES W. BEEKER, Chalmers, Ind., dredging contractor and former representative in the Indiana legislature, died recently at the age of 70 years.

J. H. BACON, engineer and architect, of Norfolk, Va., was found dead June 4 in New York City, having fallen sixteen stories to the pavement in the rear of the building where his brother Henry Bacon, architect, had offices; Mr. Bacon was 60 years of age. A graduate of Harvard in 1882, Mr. Bacon served on several river improvement projects, on portions of the Mississippi River, Cape Fear River, Missouri and Ohio Rivers and Lockwood's Folly River. In 1892 he became U. S. assistant engineer in the Florida district and had charge of many river improvement works, notably on the St. Johns River, and in military construction work at Tampa Bay and at Key West.

EDWARD C. SHANKLAND, Chicago, senior member of the firm of E. C. & R. M. Shankland, consulting engineers, died at his home on June 3. Mr. Shankland was born at Pittsburgh, Pa., Aug. 2, 1854, and after education in the public schools he was graduated from Rensselaer Polytechnic Institute in 1878. From 1878 to 1883 he was engaged on the Missouri and Mississippi River improvement work, becoming principal assistant engineer of the Leavenworth division on the Missouri River. He was then engineer of the Wrought Iron Bridge Co., Canton, Ohio, until 1889, when he went to Chicago as structural engineer for Burnham & Root, architects, becoming a member of the new firm of D. H. Burnham & Co. in 1894. During this period he was successively engineer of construction and chief engineer of works for the Columbian Exhibition of 1893. In 1898 he and his brother organized the firm of E. C. & R. M. Shankland, for consulting practice, largely in connection with steel building design. From 1911 to 1916 Mr. Shankland was a member of the Chicago Harbor & Subway Commission which carried out the project for the 3,000-ft. municipal pier and which prepared two of the many plans for an underground rapid-transit railway system. Mr. Shankland was a member of the American Society of Civil Engineers, the Western Society of Engineers and the Institution of Civil Engineers (England). This last society in 1896 awarded him the Telford gold medal.

From the Manufacturer's Point of View

A Point of Contact Between Maker and User of Construction Equipment and Materials

Asphalt Producers Report on Actual Use of Simplified Grades

Growing Acceptance of New Standards for Paving Material Indicated by Replies to "Engineering News-Record's" Inquiry

AS A RESULT of conferences participated in by producers and consumers of asphalt for road and paving work, held under the auspices of the Division of Simplified Practice of the Department of Commerce, Washington, D. C., agreement was reached last year on the 10 simplified grades (determined by the penetration test) indicated in the accompanying table. These grades became effective Jan. 1, 1924, and state, county and city highway departments were urged to specify their asphaltic material in accordance with the approved simplified scale.

To determine the extent to which consumers are conforming to the new simplified range of grades *Engineering News-Record* requested statements from all of the leading asphalt producers. The following replies were received:

STANDARD OIL CO. OF NEW YORK.
By B. L. Boye, Road Oil Dep't.

The state, county and city highway departments and other users of asphalt are practically unanimous in confining their orders to the 10 simplified grades. Occasionally some town or village will ask for a grade not included in the ten, but when their attention is called to the fact that we no longer manufacture a grade as requested by them, they usually accept the limits as defined under the simplified practice standards. We are adhering strictly to this basis and are receiving the co-operation of all users.

SUN OIL CO., *Philadelphia.*
By C. D. Campbell, Sales Manager, Hydrolene Dep't.

We have taken on a very limited amount of business for paving asphalt and road oil. I believe, however, that the highway departments are adhering closely to the 10 simplified grades agreed upon at the last year's conferences with the Division of Simplified Practice of the Department of Commerce. We are confining ouselves to only two or three soft grades of paving material.

IMPERIAL OIL, LT'D., *Toronto.*
By F. A. Hogan, Manager, Asphalt & Road Oil Dep't.

As far as Imperial Oil, Lt'd., is concerned, and on our shipments in the Dominion of Canada, we have been able to take care of all requirements with 6 grades of asphalt for paving work, including the binder used for asphalt-macadam of the penetration type. While it may be advisable to eliminate one grade, or two at least, from this schedule, we think that we are well within the limits. We are impressing upon our sales force the importance of reducing the number of grades, and we feel that this has been accomplished in our work in the Dominion.

STANDARD OIL CO. OF LOUISIANA, *New Orleans.*
By Prevost Hubbard, The Asphalt Association.

Your letter addressed to J. S. Helm, Standard Oil Co. of Louisiana, has been referred to this office with a request that we advise you that the Standard Oil Co. of Louisiana is strictly complying with the simplified grades of asphalt adopted by the conference called by the Department of Commerce last year and is furnishing only such grades to the paving trade. In general the simplified grades have been quite favorably received by engineers throughout the country and while there are some few exceptions, most of these engineers are calling for the grades adopted by the conference.

ASPHALT
Simplified Practice Recommendation, U. S. Department of Commerce

I. For construction of sheet asphalt, asphaltic concrete and asphalt macadam pavements, and also for surface treatment:

Penetration Limits

25-30	50-60	100-120
30-40	60-70	120-150
40-50	85-100	150-200

II. For joint filler for various types of construction:

Penetration Limits

30-50	60-70*
50-60*	85-100*

*These grades identical with those in (I).

I am in receipt of a letter from R. M. Hudson, of the Department of Commerce, stating that that department wishes to have an annual check-up on this proposition and asking if it will be possible for us to gather data, say for the first nine months of 1924, so that some time next October or November a meeting of the standing committee which developed the first recommendation can be called to decide whether any modifications or revisions are necessary. We think that the department's suggestion is an excellent one and will endeavor to collect the data in time for the meeting.

FREEPORT ASPHALT CO., *Freeport, Texas.*
By R. W. Stone, Vice-President and Sales Manager.

We are glad to advise you that we are heartily in accord with the work of the Division of Simplified Practice. We have now in process of being compiled a specification sheet for paving asphalts which will conform to the grades adopted. By far the larger percentage of our business is with roofing manufacturers, and we do not feel in position to advise regarding the results so far obtained in the direction of standardizing on the list of grades. We can assure you, however, that we will lose no opportunity in supporting standardization.

NEW ORLEANS REFINING CO., INC., *New Orleans.*
By Leroy M. Law, Chief Chemist.

Owing to the late spring and the consequent delay in actually starting this season's paving program in most localities, we believe that it is too early to give any definite statement of beneficial results of the simplified program. As far as we can note, however, the new specifications coming out appear to be generally in harmony with the simplified scheme, but we would prefer to wait until we have received more shipping instructions before expressing an opinion as to how the spirit of the understanding is being carried out.

BARBER ASPHALT CO., *Maurer, N. J.*
By C. N. Forrest, Manager.

It is a little too early to ascertain how generally the simplified scale of grades of asphalt for paving purposes will be followed by highway engineers and contractors The spring orders for such asphalt received so far indicate that the new scale is being favored and I feel sure that the producers are all sincere in their determination to encourage the adoption of the new scale by consumers, as far as that is possible. The use of this scale will be mutually advantageous.

STANDARD OIL CO. (New Jersey).
By Herbert Spencer, Fuel Oil Dep't.

Since the adoption of the Department of Commerce simplified grades for asphalt it has been necessary to call to the attention of quite a number of states the fact that they had agreed to this simplified practice before they would consent to change their printed specifications. However, I am glad to say that this has been accomplished in practically every case where the matter was taken up with them, and this applies to Ohio, Virginia, West Virginia, North Carolina and New Jersey. These states have either changed their printed specifications to conform to these standards or have written letters to their division engineers and to the producers authorizing shipment of grades in accordance with the limits as adopted. Unfortunately, the City of New York has not taken this same stand, although I understand from Mr. Hubbard, of the Asphalt Association,

that most of the boroughs have agreed to meet the request of the producers and have signified their intention of adopting the Department of Commerce limits and will soon issue a notice to this effect.

I believe this practice of simplified specifications on asphalt should be extended beyond merely the penetration limits, and that an effort should be made to standardize all tests called for on asphalt materials going into street or road work. Where we have gained in the paving asphalts and binders called for, we have decidedly lost ground on the cold surface application materials called for by various state and county highway departments.

If you will look over the specifications of New York, New Jersey, Pennsylvania, Virginia, North Carolina, Ohio, Michigan—to mention a few—you will note the chaotic condition in which these hot and cold surface application specifications are at the present time. The variety of grades called for imposes a serious handicap on the producer and necessitates an amount of work, both in determining what is called for and in producing the material to meet these specifications, out of all proportion to the quantity of business involved. Moreover, with various societies engaged in drafting such specifications without agreement on what is actually wanted, the problem is one which seems almost hopeless, but which I believe should be attempted through a conference similar to the one held last year in Washington. It is vitally necessary, however, that the various states either agree to attend such a conference or agree first among themselves as to what is actually required for their surface treatment materials. Producers, I am sure, will fall in line with anything which is actually required and object mainly to the multitude of grades called for by these different states.

As far as the effects of the penetration limits are concerned, we have noticed a very decided improvement over previous years. I consider the effects very satisfactory and think that within another year the particular feature of limitation on penetration grades will be even better for the producer than it has been this year.

UNION OIL CO. OF CALIFORNIA, *Los Angeles.*

By J. B. Arthur, Manager, Asphalt Sales.

The grades of asphalt selected by the standardization committee practically conform to the grades demanded by the trade in this territory, so that no educational measures were necessary on our part to restrict ordering to these grades. The various grades adopted were more than sufficient to cover the range of the materials in demand by normal practice on the Pacific Coast.

MEXICAN PETROLEUM CORP. OF LOUISIANA.

By H. T. Hulin, Superintendent.

At the present time 94 per cent of the orders we have on file are confined to the specifications recommended at the Washington conference. However, this does not so much denote a tendency on the part of the users of asphaltic products to adhere to the specifications adopted as it does to the ruling on the part of this corporation only to accept orders that are within the specifications agreed upon. While this may seem rather arbitrary and may also result in a loss of business to us, we believe that it will eventually be a benefit not only to us but to other producers of asphalt, and to consumers as well, for the reason that it will have a tendency to cause the trade to adopt the simplified schedule of specifications.

STANDARD OIL CO. (California), *San Francisco.*

By J. K. Firth, Jr., Manager, Asphalt Division.

Our asphalt sales for paving work in the western states fall generally within the limits of a few of the simplified grades. Asphaltic concrete is generally made using our "D" grade asphalt, penetration 40 to 50 or 50 to 60, which, it may be seen, is covered by two of the mentioned simplified grades. Asphalt macadam requires a much higher penetration of asphalt, and is covered by our "E" grade, which is marketed in the following penetration ranges: 91-110; 111-130; 131-150; and 151-250.

It is difficult to state just how paving in general has been affected by these simplified grades. Our shipments consigned to paving contractors fall very closely within the limits of these simplified grades.

ATLANTIC REFINING CO., *Philadelphia.*

By Irvin K. Giles, Chief Chemist.

Our Sales Department informs me that the state highway departments have followed the simplified practice recommendations, but that there are quite a number of cities and general contractors who seem to be uninformed as to the revised list. All things considered, it appears that there are fewer grades of the harder material being called for this year than formerly.

Mixer Manufacturers Join New A.G.C. Bureau

Manufacturers of concrete mixers, meeting in Chicago, May 1, as noted in this journal May 15, p. 918, passed a resolution recommending that a mixer manufacturers' group be established in the Affiliated Bureau of the Associated General Contractors and that a committee of manufacturers be appointed to canvass the entire mixer industry. This committee was appointed as follows:

Philip Koehring, Koehring Co., chairman; R. Bowen, T. L. Smith Co.; W. B. Knickerbocker, Knickerbocker Co.; R. C. Weller, Marsh-Capron Co.; A. C. Scannell, Archer Iron Works.

Those companies which desired definitely to affiliate with the bureau are: Archer Iron Works, Knickerbocker Co., Atlas Engineering Co., T. L. Smith Co., Brook Co., Marsh-Capron Co., Lakewood Engineering Co. and Koehring Co. Representatives of other companies who were not authorized to commit those companies in this matter, expressed a belief that they would undoubtedly join the bureau.

Business Notes

CHICAGO PNEUMATIC TOOL CO., New York, has removed its Los Angeles branch, sales and service headquarters, from 307 East Third St., to the company's new building at 655 Santa Fe Ave. The change has been made necessary by the expanding business handled from the Los Angeles branch, which serves southern California, southeastern Nevada, Arizona, lower California and Sonora, Mexico.

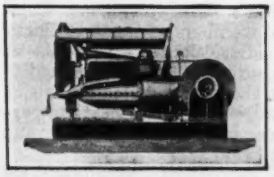
CONCRETE REINFORCING & ENGINEERING CO., Cleveland, has changed its name to the Rivet-Grip Steel Co. The company manufactures light steel trusses for use as joists in fire-proof floor construction.

JOHN T. HORTON CO., INC., New York, manufacturer of derricks, winches and cableways, announces the death on May 25 of its president, John T. Horton.

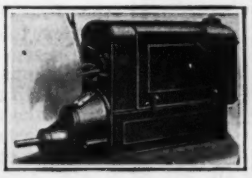
Equipment and Materials

Gasoline Power Unit for Varied Construction Uses

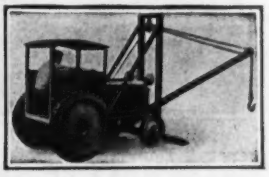
Designed for a variety of uses in construction and industrial work, a 25-hp. gasoline power unit, equipped with a Fordson engine, together with auxiliary features for special uses, has been placed on the market by the newly organized Otis Engine Corporation, of New York. For contractors, one of the new products is the Otis three-speed reversible hoist, made in single-drum and double-drum models. For the single-drum unit the rating given by the manufacturer is a 3,500-lb. pull on a single line at 150 ft. per minute hoisting speed. The standard drum is 14 in. in diameter, 15 in. long, and has end flanges 26 in. in diameter, giving a rope-winding capacity of 1,900 ft. of ¾-in. cable. Depending on the load, the speed range is between 500 and 20 ft. per minute. The single-drum hoisting unit weighs complete 4,000 lb., is 44 in. wide, 54 in. high and 86 in. long; the double-drum unit weighs 5,000 lb., is 68 in. wide, and of the same length and height as the single-drum model.



HOISTING ENGINE



POWER UNIT



TRAVELING CRANE

Winch heads are an auxiliary feature on both types. The hoisting engines are mounted on structural steel box sub-bases with 4 x 6-in. timber skids fastened to the side channels.

Using the same power equipment, the company manufactures a mobile 1-ton swing crane and a ½-ton traveling crane and hoist; in both cases the Fordson engine is mounted on rubber-tired wheels. The swing crane, with reversible single-drum hoist, is fitted with an 8-ft. boom with a lateral operating range of 150 deg.; for loads lighter than 1 ton booms up to 16 ft. may be used. For the traveling crane an A-frame construction is mounted at the front end of the Fordson engine and held rigidly by a tie-rod. It has a reach of 4 ft. and will raise a load to a height of 11 ft., sufficient to clear the side of a motor truck.

For miscellaneous power requirements the Otis power unit, without attachments, is provided. Here also the 4-cylinder Fordson engine is the prime mover and the rating of the plant is 25 hp. Speed reduction is accomplished through gears or worms, the speed range being from 10 to 3,500 r.p.m. The control is by a single lever and the drive through a multiple disc friction clutch running in oil. The unit is completely inclosed in a sheet-metal housing and has a variety of power take-offs, both angular and straight line.

The Otis line of equipment includes, also, a tractor-driven rotary broom street sweeper and a rail car.

Pneumatic Clay Digger

The new Thon pneumatic clay digger or spade of the Independent Pneumatic Tool Co., Chicago, shown herewith, is designed particularly for excavating clay in tunnel, trench, deep-well and

general foundation work. It strikes about 800 blows per minute and can be handled readily by unskilled labor. In the trench illustrated, for 18-in. water mains, two men with pneumatic diggers kept four men busy shoveling the clay thus removed. In foundation piers at the Chicago Union Station these tools were used in belling the bottoms of the shafts to enlarge the bearing area on a hardpan stratum. The spade can be replaced with chisel to cut off wood sheeting, a drill to bore holes in concrete, or a bull-point chisel to break up or cut holes in concrete.

Business Side of Construction

FACTS AND EVENTS THAT AFFECT COST AND VOLUME

Business Briefs

Call money ranged from 2@2½ per cent, June 9.

Time money loaned for short terms at 3¾ per cent; longer periods at 4 per cent.

Commercial paper rates for all names —4½@4¾ per cent.

Foreign exchange: (Demand)

	This Week	Last Week	Year Ago
Sterling	$4.30⅞	4.31⅛	$4.61½
Franc	0.0507½	0.0509½	0.0644½
Lira	0.0434½	0.0434½	0.0465

American Consuming Power Close to Productive Capacity

In a recent speech before the United States Chamber of Commerce, Julius H. Barnes, president, gave some facts pertaining to production and consumption of basic materials used in construction.

The extensiveness of the American industrial mechanism may be visualized when it is realized that the United States has about 6 per cent of the world population and yet annually one-half of the annual output of the basic materials of the world is converted by our industries into usable forms. The following table shows production and consumption of some of the essential materials by the United States:

	Produces Per Cent	Consumes Per Cent
Coal	43	42
Iron	54	53
Steel	64	57
Copper	49	44
Petroleum	64	72
Timber	52	51

The growth of the nation's productive capacity is indicated by the following figures, giving the production per man-day in 1781 and 1923:

Iron, lb.	500	5,000
Lumber, ft.	100	750
Nails, lb.	5	500
Coal, ton	¼	4

Construction Called Balance Wheel of Industry

E. E. Hunt, executive secretary of the President's Unemployment Commission, told members of the Associated Engineering Societies of St. Louis at their annual dinner, June 4, that building activity should not be the slave of seasons because American weather permits construction work almost every month of the year.

He also advocated municipalities doing their construction work during periods of depression in usual building channels.

"Only by control of the peak periods in the business boom can we hope to prevent depression," he said, "which is a handicap to architects, engineers, and other specialists in the building industry. Defective management is the greatest cause of waste of time and men, but economic conditions also have a certain effect on waste.

(*Continued on p. 1042*)

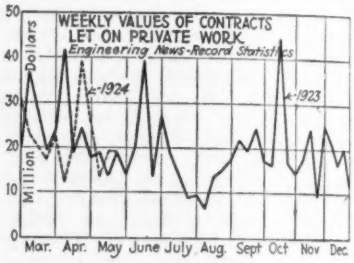
Value of Public Works Contracts Greater This Year Than Last

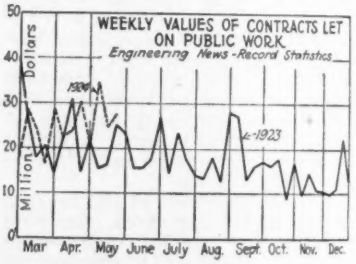
The total money value of contracts awarded on both private and public work, for the week ended June 5, was greater than that for the corresponding period in 1923.

Compared with the week preceding, however, public lettings showed a heavy gain and private awards a decrease in total money value.

The average weekly total of public awards throughout the year 1923 was $17,000,000, while the average of weekly lettings thus far this year is $19,000,000.

The weekly average for private awards in 1923 was $19,000,000, against $21,000,000 for the first twenty-three weeks of this year.





Minimum costs observed in the Construction News Section of *Engineering News-Record* on each class of construction are as follows: Waterworks and excavations, $15,000; other public works, $25,000; industrial construction, $40,000 and commercial buildings, $150,000.

The five high peaks in the accompanying chart of private contracts, four in 1923 and one in May of this year, may be explained by the following list of exceptionally large awards: March 29, 1923, railroad and telegraph system, Montana, 330-mi., $12,000,000; April 26, 1923, Texas sugar refinery, Texas City, $5,500,000; station, B. & A. R.R., Springfield, Mass., $4,000,000; bank, St. Louis, Mo., $3,000,000; floating dry dock, North Vancouver, B. C., $3,750,000; June, 28, 1923, Telephone Exchange, New York, N. Y., $11,250,000; two office buildings in Chicago, Ill., $7,000,000 and $5,500,000, respectively; office building, New York, $5,000,000; Oct. 18, 1923, Public Service power station, Kearny, N. J., $20,000,000 and May 8, 1924, the McCorkle pipe line, San Antonio, Tex., $15,000,000.

Big Drop in Iron and Steel Output—Lumber Slower—Gains in Brick and Cement Production

Warehouse Stocks Small—Mill Deliveries Prompt—Some Importation of Glass, Brick, Cement and C.-I. Pipe

Iron—Pig-iron production during May amounted to 2,615,000 tons, against 3,233,000 for the preceding month. This is equivalent to a drop of over 23,000 tons in the daily output during the month and represents the heaviest cut in production since the steel strike in 1919.

Steel—The May total of 2,628,261 tons of steel ingots for the entire country, as calculated by the American Iron and Steel Institute, is but little more than half the output for May of last year. May, 1923 was the record month of that year with a total output of 4,216,355 tons. The April, 1924, output was 21 per cent greater than that for the month just past. Monthly production figures for the first five months of this year and last, are compared in the following table:

	1924 Tons	1923 Tons
Jan.	3,633,639	3,841,095
Feb.	3,809,185	3,471,843
Mar.	4,187,942	4,066,680
Apr.	3,333,535	3,963,736
May	2,628,261	4,216,355
5 mos.	17,592,562	19,559,709

Brick—Orders on books of the Common Brick Manufacturers' Association of America, as of May 1, showed a gain of 14 per cent over the month preceding. With this increase in demand, however, there were fewer burned brick on hand than on April 1, 1924. There are more brick, burned and unburned, now at yards throughout the country than there were a year ago. The current situation compared with a month ago and one year ago, is shown in the following tabulation:

1923	Burned Brick on Hand M	Unburned Brick on Hand M	Brick Moved from Yards During Month M	Orders on Books M
May 1	150,316	49,129	.....	464,604
1924				
Jan. 1	313,696	55,646	66,199	266,914
Feb. 1	306,503	38,349	45,762	272,403
Mar. 1	282,813	39,695	104,663	263,219
Apr. 1	250,118	46,226	142,197	335,926
May 1	238,703	58,972	169,855	382,778

Lumber—The national lumber movement for the week ending May 31, is one of decreased production, shipments and demand, according to the National Lumber Manufacturers' Association. The same is true of a comparison with the corresponding week in 1923. The following figures compare the lumber movement for the first twenty-two weeks of 1924 with the same period last year:

	Production Ft. B.M.	Shipments Ft. B.M.	Orders Ft. B.M.
1924	5,210,046,828	5,157,843,213	4,837,861,536
1923	5,092,058,937	5,540,474,723	5,315,854,525
1924 Increase	117,987,891	.....	.....
1924 Decrease	.....	382,631,510	477,992,989

The following is a weekly comparison of lumber movements:

	Week Ending May 31 Ft. B.M.	Corresponding Week in 1923 Ft. B.M.	Preceding Week in 1924 Ft. B.M.
Production	225,424,327	257,031,837	245,975,795
Shipments	206,423,684	253,425,282	229,284,730
Orders	177,647,933	194,831,032	185,754,303

Cement—The output for the first four months of the current year was over 5 per cent greater than that for the corresponding period in 1923. Shipments, however, fell off about 5 per cent, leaving stocks on hand at mills throughout the country, 49 per cent greater than the quantity available at this time last year. The cement situation for the first four months of this year compared with last, is shown as follows:

1923	Production Barrels	Shipments Barrels	Stocks on Hand at End of Month Barrels
January	7,990,000	5,628,000	11,477,000
February	8,210,000	6,090,000	13,596,000
March	9,880,000	10,326,000	13,045,000
April	11,359,000	12,954,000	11,463,000
4 mos.	37,439,000	34,998,000	.....
1924			
January	8,788,000	5,210,000	14,155,000
February	8,588,000	5,933,000	16,815,000
March	10,370,000	8,995,000	18,189,000
April	11,726,000	12,771,000	17,144,000
4 mos.	39,472,000	32,909,000	.....

While there is no actual shortage of any of the basic materials, dealers' warehouse stocks are being kept as low as possible because of the danger of price depreciation on heavy inventories. Mill shipments are unusually prompt; much buying, ordinarily done at dealers' warehouse, is therefore being transacted at the mill.

Importations of window glass, brick, cement and c.-i. pipe are gradually increasing. Most of the plate glass and brick are coming into the New York district, while the heaviest cement shipments are being consigned to the West Coast. The Middle Atlantic states and California are both taking foreign c.-i. pipe.

Brooklyn Subway Project for $3,948,831 Rejected

The rejection by the Board of Estimate of the contract awarded last month for the Brooklyn crosstown subway, will necessitate the deduction of $3,948,831 from the total of contracts awarded during May as published in

CONDITIONS OF MATERIALS STOCKS IN IMPORTANT CENTERS

Stocks on hand in approximate figures, example: (Common brick, Denver, 360,000); time required for delivery of carload lots to city job, example: (Hollow tile, Atlanta, deliveries take 4 @ 5 days); and stocks on hand in general terms, example: (Sewer pipe, Cincinnati, plenty, all sizes).

	San Francisco	Denver	Minneapolis	Detroit	Cincinnati	New Orleans	Atlanta	Philadelphia	New York
Sewer pipe	Stocks large; all sizes	Reserves low; behind demand	Small stocks in city; mill del. prompt	Moderate supply in local yards	Plenty all sizes	Plentiful; demand light	Del. take 4 to 5 days from mill	About 120 cars	Dealers' stocks usually small; mill del. prompt
Cement	Fairly well supplied	Ample for current demand	Warehouse stocks sufficient for immediate demand; mill del. prompt	Plenty at local and nearby mills	Readily obtainable	Abundent; small demand	About 75 cars	Small stocks	More on hand than year ago
Lime	Stocks in fair shape	Sufficient	Small quantity in dealers' stocks; 24 hr. del. from kilns	Prompt shipments from Mich. and Ohio kilns	Adequate supplies	Sufficient	About 70 cars on sidings	Supply sufficient to meet demand	Dealers' stocks usually small; mill del. prompt
Common brick	Large reserves in yards	Only 360,000 on hand	Del. prompt from nearby yards.	Ample supply in local yards	Plentiful	Adequate; fair demand	Plenty	Local yards well supplied	Ample supply due to steel erectors' strike
Hollow tile	Fairly well supplied	Reserves low; behind demand	No large stocks carried by dealers	Comparatively small stocks	No trouble in filling orders	Normal; demand light	Del. take 4 to 5 days	No dealers' stocks; prompt del. from kilns	Dealers' stocks usually small; prompt mill del. by rail and water
Lumber	Very large reserves	Normal	Yard stocks normal	Average stocks in local yards	Adequate stocks in dealers' yards	Pine stocks limited	Plenty	Complete stocks in yards	Stocks heavier than year ago
Asphalt	Very large native reserves	10 cars on sidings	Small quantity stored; del. take 7 to 10 days	Enough to meet demands	Plenty	150 tons; 1,200 needed	About 75 cars	No surplus carried	Heavy reserves nearby
Structural steel	Fairly well supplied; demand good	Warehouses well stocked	Ample warehouse reserves	No shortage	Demand more active in warehouses but in small lots	Warehouse stocks plentiful	About 5 cars	Small warehouse stocks; mill del. prompt	Warehouse demand confined to small tonnage lots

Engineering News-Record, issue of June 5.

This will change the grand total of all contracts above a certain minimum on each class of construction, to $253,716,000, against $257,665,000.

The *Engineering News-Record* Construction Volume Index Number will accordingly be changed to read 203 instead of 206 as published on p. 997 of the June 5 issue.

Construction Called Balance Wheel of Industry

(*Continued from p. 1040*)

"Construction is the balance wheel of industry. The day of engineers and economists has dawned. I do not wish to appear as a prophet, but I think the future will see a greater number of engineers directing the affairs of the state instead of the many lawyers and orators."

Large Contracts Let During Week

Among the week's announcements of contracts awarded in Construction News, pp. 343 to 356, are the following:

Apartment, Buffalo, N. Y., to H. L. Stevens, New York City, $2,000,000.

Hotel, Birmingham, Ala., to S.-B. Construction Co., $1,250,000.

Water and sewerage systems in 32 cities in Peru to Foundation Co., New York City, $50,000,000.

Weekly Construction Market

THIS limited price list is published weekly for the purpose of giving current prices on the principal construction materials, and of noting important price changes on the less important materials. Moreover, only the chief cities are quoted.

Valuable suggestions on costs of work can be had by noting actual biddings as reported in our Construction News section.

The first issue of each month carries complete quotations for all construction materials and for the important cities. The last complete list will be found in the issue of June 5, the next, on July 3.

Steel Products:	New York	Atlanta	Dallas	Chicago	Minneapolis	Denver	San Francisco	Seattle	Montreal
Structural shapes, 100 lb.	$3.59	$3.80	$4.40	$3.30	$3.55	$4.20	$3.60	$3.60	$4.20
Structural rivets, 100 lb.	4.00	4.35	4.90	3.75	4.00	5.15	5.00	4.25	5.00
Reinforcing bars, ¾ in. up, 100 lb.	3.49	3.30	3.38	3.20	3.45	3.85	3.65	3.50	—2.70
Steel pipe, black, 2½ to 6 in. lap, discount	44%	54.2%	43%	47%	55.05%	36%	36.80@49.10%	45%	47.43
Cast-iron pipe, 6 in. and over, ton	61.60@63.60	—52.00	+61.50	57.20@60.20	—58.50	68.00	59.00	65.00	60.00
Concreting Material:									
Cement without bags, bbl.	2.50@2.60	2.35	2.05	2.20	2.42	2.84	2.61	2.90	—1.90
Gravel, ¾ in., cu.yd.	1.75	2.00	2.38	2.00	1.85	1.90	2.15	1.50	—1.25
Sand, cu.yd.	—1.00	1.40	2.00	2.00	1.25	1.00	1.50	1.50	1.25
Crushed stone, ¾ in., cu.yd.	1.85	2.00	2.83	2.00	—1.75	3.50	2.15	3.00	2.25
Miscellaneous:									
Pine, 3x12 to 12x12, 20 ft. and under, M.ft.	—59.00	40.00	60.22	—51.50	38.75	—37.75	36.00	25.00	—60.00
Lime, finishing, hydrated, ton	18.20	22.50	20.00	20.00	25.50	24.00	22.60	24.00	21.00
Lime common, lump, per bbl.	3.00@3.25	1.65	1.85	1.50	1.40	2.70	2.10	2.80	10.50
Common brick, delivered, 1,000	23.65	12.00	14.10	11.00	16.00	12.00	15.50	15.00	17.50
Hollow building tile, 4x12x12, per block	Not used	.091	.11	.064	+.08	.075	.....	.11	.12
Hollow partition tile 4x12x12, per block	.1162	.091	.11	.....	.....	.075	.108	.11	.....
Linseed oil, raw, 5 bbl. lots, gal.	.98	+1.02	1.14	.94	1.08	+1.18	1.05	1.13	1.23
Common Labor:									
Common labor, union, hour	.75	.30	.....	.....	.....	.50@.55	.55	.62½	.....
Common labor, non-union, hour	....	.25	.30@.50	.82½	.50@.55	.35@.50	.50	.62½	.25

Explanation of Prices—Prices are to contractors in carload lots unless other quantities are specified. Increases or decreases from previous quotations are indicated by + or — signs. For steel pipe, the prevailing discount from list price is given; 45-5% means a discount of 45 and 5 per cent.

New York quotations delivered, except sand, gravel and crushed stone, alongside dock; common lump lime, in 280-lb. bbl. net, and hydrated lime f.o.b. cars; tile "on trucks"; linseed oil and cast-iron pipe f.o.b.

Labor—Concrete laborers' rate, 93¾c.; building laborers (pick and shovel men) 75c. per hr.

Chicago quotes hydrated lime in 50-lb. bags; common lump lime per 180-lb. net. Lumber delivered on job.

Minneapolis quotes on fir instead of pine. Brick, sand and hollow tile delivered. Cement on cars. Gravel and crushed stone quoted at pit. We quote on brown lime per 180-lb. net; white is $1.80 for Kelly Island and $1.70 for Sheboygan. Common labor not organized.

Denver quotes on fir instead of pine. Cement "on tracks"; gravel and sand at pit; stone on cars; lime, brick, hollow tile and lumber on job. Tile price is at warehouse. Linseed oil, delivered, in iron bbl. Common lump lime per 180-lb. net.

Atlanta quotes sand, stone and gravel per ton instead of cu.yd. Common lump lime per 180-lb. net.

Dallas quotes lime per 180-lb. bbl. Steel, cement, cast-iron pipe and crushed stone f.o.b. cars, other materials delivered.

San Francisco quotes on Heath tile, size 5½ x 8 x 11½. Prices are all f.o.b. warehouses except C. I. pipe, which is mill price plus freight to railway depot at any terminal. Common lump lime per 180-lb. net. Lumber prices are to dealers in yards at San Francisco, for No. 1 fir, common.

Seattle quotes on Douglas fir (delivered) instead of pine. Lump finishing lime per 180-lb. net. Hollow building tile delivered. Hydrated lime in paper sacks. Sand and gravel at bunkers.

Montreal quotes on pine lumber. Sand, stone, gravel and lump lime per ton. Stone, lime and tile are delivered; sand, gravel and cement on siding; brick f.o.b. plant; steel and pipe at warehouse. Hollow tile per ft. Cement price is in Canadian funds (the Canadian dollar stands at 98.34). Bag charge is 80c. per bbl. Discount of 10c. per bbl. for payment within 20 days from date of shipment. Steel pipe per 100 ft. net; 3-in., $47.43; 6-in. $119.

Conditions Affecting the Market

The downward trend in construction cost, evident during the past two weeks, has centered mainly in concreting materials and lumber in the United States and in cement and steel bars in Eastern Canada.

Slight advances here and there throughout the country in such materials as c.-i. pipe, hollow tile and linseed oil, show that a protracted price decline is by no means probable.

Resistance to lower wages or actual demands for higher schedules will also serve to prevent an abrupt decline in materials prices.

The New York iron workers have practically won the strike in which they demanded $12 per day against the former rate of $10.50. New Orleans electricians have rejected the employers' offer of 90c. per hr., against their demand for $1.35. In Chicago, however, the Building Trades Council voted, June 7, to demand no wage advance but instead to devote all energies to the establishment of closed shop conditions.

While there is a reported slowing down in residential construction of the expensive type in the New York district, there is still an acute demand for cheaper housing facilities in the outlying districts.

Compared with a year ago, May building permits gained considerably in St. Louis, Mo., and Tacoma, Wash.

Telegraphic reports to *Engineering News-Record* from Seattle, state that May building permits were the largest ever experienced in that city. The reports also state that the favorable action taken by Congress on appropriations, continues to have a good effect on road building throughout the Northwest and that the balance of the season promises to be unusually active.

Improvement in demand for steel plates has been reported during the past week; the present price is $2@$2.25 per 100 lb., Pittsburgh. Reinforcing bars are quoted now at $2.20@$2.25 and shapes at $2.10@$2.25 per 100 lb. at mill. The current range of steel prices is a trifle below the level of May 29.

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